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CONTENTS, VOL. 126, PART IV

PAGE

BUILDER OF THE NEWPORT TOWER
By Edward Adams Richardson 1
Discussion:
By Edward Adams Richardson 23
HIGHWAY LOCATION AND DESIGN BY PHOTOGRAMMETRIC-ELECTRONIC COMPUTER
By S. E. Ridge 27
COLUMBIA BASIN STREAMFLOW ROUTING BY COMPUTER
By David M. Rockwood 32
Discussion:
By Willard M. Snyder 46
David M. Rockwood 54
DESIGN AND CONSTRUCTION OF NAVY'S LARGEST DRYDOCK
By S. P. Zola and P. B. Boothe 57
HYDRODYNAMIC WAVE PRESSURE ON BREAKWATERS
By M. A. Gouda 87
APRON DESIGN FOR LIGHT AIRPLANES
By Kenneth K. Wilde 100
AERIAL PHOTOGRAPHY IN ARCTIC AND SUBARCTIC ENGINEERING
By Robert E. Frost 116
EVALUATION OF ALTERNATIVE SUBWAY ROUTES
By B. A. Griffith and H. G. von Cube 146
EPOXY ASPHALT CONCRETE FOR AIRFIELD PAVEMENTS
By W. C. Simpson, H. J. Sommer, R. L. Griffin, and T. K. Miles 156
NAVIGATION MODEL STUDIES OF NEW OHIO RIVER LOCKS
By Eugene P. Fortson, Jr., and George B. Fenwick 171
CANADIAN SECTION OF THE ST. LAWRENCE SEAWAY
By Lawrence H. Burpee 184
AIRPHOTO INTERPRETATION FOR AIRFIELD SITE LOCATION
By James H. McLerran 210
CORRELATION OF VEHICLE DESIGN AND HIGHWAY DESIGN
By Richard A. Haber and David K. Witheford 228
ANALYSIS OF LOAD DISTRIBUTION IN THE PILES OF PIERS
By M. A. Gouda 249
SURVEYS AT COUGAR RESERVOIR
By Charles N. Oros 272
SDR SURVEY FOR PROPOSED CHESAPEAKE BAY CROSSING
By Walter C. Beckmann, Charles L. Drake, and George H. Sutton 278
SOUTHWEST PASS-MISSISSIPPI RIVER 40-FT. CHANNEL
By Austin B. Smith 291

PAGE

PLAN FOR CLOSURE OF OLD RIVER
By George M. Cookson 311
SHARK RIVER INLET SAND BY-PASSING PROJECT
By W. Mack Angas 331
PNEUMATIC BARRIER AGAINST SALT WATER INTRUSION
By Ian Larsen 350
STUDIES OF A CHANNEL THROUGH PADRE ISLAND, TEXAS
By E. A. Hansen 363
FREEWAY SPACING IN AN URBAN FREEWAY SYSTEM
By James M. Peterson 383
ARKANSAS RIVER PLAN
By William Whipple 394
ELECTRONIC COMPUTER IN HIGHWAY ENGINEERING
By L. R. Schureman 408
Discussion:
By Charles E. Carlson 431
L. R. Schureman 432
COUNTY MOTOR VEHICLE TRANSPORTATION SYSTEM PLAN
By Victor W. Sauer 434
BANK PROTECTION ON CENTRAL VALLEY STREAMS
By R. W. Barsdale 458
HYDROLOGY AND FLOOD CONTROL FEATURES OF OROVILLE DAM
By Joseph I. Burns, Kenneth B. Mayo, and William A. Arvola 473
LABORATORY INVESTIGATION OF RUBBLE-MOUND BREAKWATERS
By Robert Y. Hudson 492
Discussion:
By Jose Reis de Carvalho and Daniel Vera-Cruz 520
William H. Booth, Jr. 523
Francis B. Slichter 525
Leland B. Jones 526
Omar J. Lillevang 528
Thorndike Saville, Jr. 531
Robert Y. Hudson 536
DESIGN OF SEA-WALLS AND BREAKWATERS
By Ira A. Hunt, Jr. 542
CASE OF CRITICAL SURGING OF A MOORED SHIP
By Basil W. Wilson 571
Discussion:
By W. C. Q. Joosting 590
Basil W. Wilson 598
SPACING OF INTERCHANGES ON FREEWAYS IN URBAN AREAS
By Jack E. Leisch 604
Discussion:
By Gerald D. Love 617
USE OF MATHEMATICAL MODELS IN ESTIMATING TRAVEL
By Alan M. Voorhees 620

PAGE

PREDICTION OF MAXIMUM PRACTICAL BERTH OCCUPANCY
By Thomas J. Fratar, Alvin S. Goodman, and Austin E. Brant, Jr. 632
Discussion:
By Donald C. Hill 642
Thomas J. Fratar, Alvin S. Goodman, and Austin E. Brant, Jr. 642

APPROACH TO THE STUDY OF OVERSEAS TRAFFIC POTENTIALS
By Wilfred G. McLennan 644

PORT STRUCTURES IN LONG BEACH HARBOR
By Bob N. Hoffmaster, Cort T. Johnson, and Charles L. Vickers, Jr. 661

SAN DIEGO TRANSPORTATION PLANNING STUDIES
By Edward M. Hall 683

ROADBEDS ON HIGHWAYS AND AIRPORT RUNWAYS
By Ira B. Mullis 699
Discussion:
By Jacob Feld 726
W. H. Campen, L. G. Erickson, and J. R. Smith 727
Ira B. Mullis 736

DEVELOPING ALTERNATIVE URBAN TRANSPORTATION SYSTEMS
By William R. McConochie 750
Discussion:
By Kenneth M. Hoover 767
S. D. Forsythe 768
Nathan Cherniack 769
William R. McConochie 771

SHOCK PRESSURES EXERTED BY BREAKING WAVES ON BREAKWATERS
By Shoshichiro Nagai 772

ENGINEERING LAW APPLIED TO HIGHWAY DRAINAGE
By R. Robinson Rowe 810

NATURAL BY-PASSING OF SAND AT COASTAL INLETS
By Per Bruun and F. Gerritsen 823
Discussion:
By J. R. Bowman 851
Per Bruun and F. Gerritsen 852

ELECTRIC ANALOG MODEL OF A TIDAL ESTUARY
By H. A. Einstein and James A. Harder 855
Discussion:
By G. B. Fenwick 868
H. A. Einstein and James A. Harder 869

LONGITUDINAL DISTRIBUTION OF WHEEL LOADS ON A RUNWAY
By R. W. Smith and R. Horonjeff 870
Discussion:
By R. G. Ahlvin 882

FOREWORD

ASCE TRANSACTIONS, 1961, Part IV, contains, nominally, all papers published in the Journals of the Air Transport, City Planning, Highway, Pipeline, Surveying and Mapping, and Waterways and Harbors Divisions, Proceedings of the American Society of Civil Engineers. The following papers were not included in this part because discussion was not complete when the volume was closed:

JOURNAL OF THE AIR TRANSPORT DIVISION

May 1960

Reinforced Concrete Pavements for Airports
by August W. Compton (Proc. Paper 2460)

JOURNAL OF THE CITY PLANNING DIVISION

May 1960

Public Transit Improvements in Toronto
by W. F. Irvin (Proc. Paper 2465)[a]

Planning Activity in the Cleveland Region
by Stephen A. Kaufman (Proc. Paper 2492)[a]

JOURNAL OF THE HIGHWAY DIVISION

March 1960

Highway Research
Progress Report of the Committee on Research of the Highway Division (Proc. Paper 2412)[c]

June 1960

Analyzing and Projecting Travel Data
by Wilbur S. Smith (Proc. Paper 2513)

September 1960

High Temperature Effects on Bituminous Mixes
by William H. Gotolski (Proc. Paper 2592)

Traffic Behavior and Freeway Ramp Design
by Charles Pinnell and Charles J. Keese (Proc. Paper 2604)

December 1960

LTS Design of Continuously Reinforced Concrete Pavement
by Benjamin F. McCullough and William B. Ledbetter (Proc. Paper 2677)

JOURNAL OF THE PIPELINE DIVISION

There were no issues of this Journal published during 1960.

JOURNAL OF THE SURVEYING AND MAPPING DIVISION

February 1960

Self-Checking Method of Computing Curve Elevations
by Charles M. Lamont (Proc. Paper 2365)[a]

The Calculating Machine in Coordinate Geometry
by Bela Vadasz (Proc. Paper 2367)[a]

Electric Analog for Triangulation Adjustment
by Hsuan-Loh Su (Proc. Paper 2370)[a]

Airborne Surveying with Doppler Systems
by L. R. Chapman (Proc. Paper 2378)[a]

Intersection of Straight Line with Spiral
by T. F. Hickerson (Proc. Paper 2379)[a]

Surveying in the Civil Engineering Curriculum
by Kenneth S. Curtis (Proc. Paper 2386)[a]

Civil Engineering Curriculum for Federal Surveying and Mapping
by G. C. Tewinkel (Proc. Paper 2387)[a]

Topographic Mapping in Alaska
by Reynold E. Isto (Proc. Paper 2388)[a]

July 1960

Highway and Bridge Surveys: Analytic Geometry in Highway Design and Layout
Progress Report of the Committee on Engineering Surveys of the Surveying and Mapping Division (Proc. Paper 2548)[c]

JOURNAL OF THE WATERWAYS AND HARBORS DIVISION

March 1960

Transmission of Waves Past a Rigid Vertical Thin Barrier
by Robert L. Wiegel (Proc. Paper 2413)[a]

Waves in Navigation Canals due to Lock Filling
by Hans-Werner Partenscky (Proc. Paper 2422)

Mean Direction of Waves and Wave Energy
by Omar J. Lillevang (Proc. Paper 2423)

June 1960

Breakwaters in the Hawaiian Islands
by Robert Q. Palmer (Proc. Paper 2507)

September 1960

September 1960

Tetrapods and Other Precast Blocks for Breakwaters
by P. Danel, E. Chapus, and R. Dhaille (Proc. Paper 2590)

Design of Inlets for Texas Coastal Fisheries
by H. P. Carothers and Homer C. Innis (Proc. Paper 2603)

November 1960

Russian River Channel Works
by Israel H. Steinberg (Proc. Paper 2647)

Sacramento River Deep Water Ship Channel: Functional Planning
by Amalio Gomez (Proc. Paper 2649)

Design Considerations for California Marinas
by James W. Dunham (Proc. Paper 2658)

In the foregoing list, the symbol [a] denotes a paper cleared for publication in the Journals prior to December 1, 1959, which will not be published in Transactions. The symbol [b] is used to signify papers that were part of Symposia that were not completed in time to publish the entire group in Transactions. The symbol [c] represents a committee report that will not be included in Transactions.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3091

BUILDER OF THE NEWPORT TOWER

By Edward Adams Richardson[1]

With Discussion by Edward Adams Richardson

SYNOPSIS

The Newport Tower, of archaeological interest, is investigated with regard to structural design and to determine possible reasons for the window and fireplace arrangement. The design proves adequate, by modern standards, for a particular church structure, while the windows and fireplace form a sophisticated signalling and ship guidance system characteristic of the 14th century.

INTRODUCTION

The problem of the Old Stone Tower, or Old Stone Mill at Newport, R. I., has given rise to a very extensive literature. It seems important that some attention be given to the builder of the tower, whoever he may have been, whenever he may have lived. As the tower itself is that man's monument and his work, it is conceivable that the tower itself can be questioned effectively through a search for the knowledge and skill which the builder must have possessed. Studies of the history of the site should be utilized.

As some readers may be unacquainted with the relic, a few words of introduction are in order (Fig. 1). In Touro Park, just south of Mill Street, which leads up from the Jamestown Ferry, there stands a rude structure of rubble masonry in lime mortar. It is not particularly impressive at a casual glance. A substantially cylindrical tower about 23 ft. in outer diameter, about 26 ft. in overall height, the upper part a solid wall with a few window and port

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Note.—Published essentially as printed here, in February, 1960, in the Journal of the Surveying and Mapping Division, as Proceedings Paper 2383. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Publications Dept., Bethlehem Steel Co., Bethlehem, Pa.,

openings which is carried, in turn, on semicircular arches, and these, in turn, on eight circular columns. The roof and floors, known to have been of wood, have disappeared. The plaster inside and outside is almost completely gone. Holes intended for large floor beams occur at about the level of springing of the arches; others for a second floor may be seen. On the east side there is an old fireplace built into the wall. Two flue holes proceed upwards from the corners thereof inside the wall to discharge through small ports in the outer wall surface. Evidence of a stairway from one floor to another may be seen, together with various slots for table tops and cubby-holes for closet purposes. The ground at the base of the tower has an elevation approximately 84 ft. above the mean sea level.

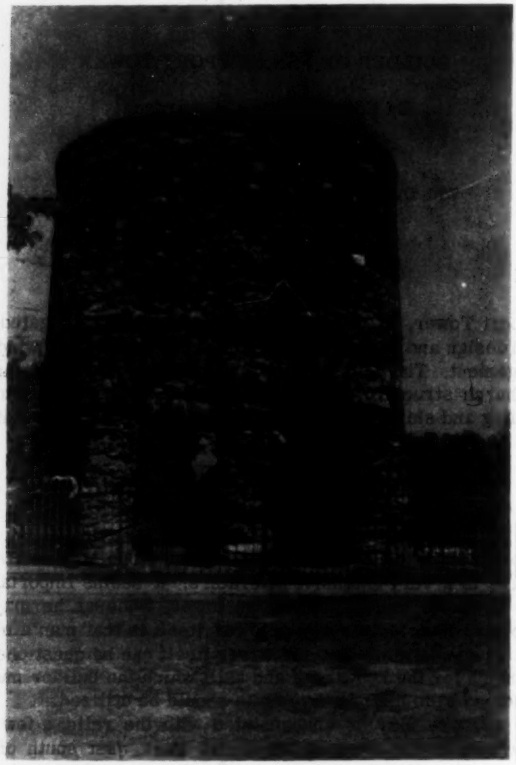

FIG. 1.—THE OLD STONE TOWER AS AT PRESENT, LOOKING EAST.

The tower itself once had views around much of the horizon. The columns must have been intended to carry at least the roof of a one-story encircling structure, for the outer sides thereof have caps adapted to such use and the arch load is imposed inside the column center lines. Further details will be supplied as they are needed.

HISTORICAL BACKGROUND

Those desiring to study the various hypotheses regarding the tower and its possible origin may read profitably and with enjoyment the four books which follow. These, in turn, list extensive references to other books and papers. A brief study may be found in "The Old Stone Mill" (1955) by Herbert Olin Brigham. Philip Ainsworth Means has written a comprehensive historical study and discussed various hypotheses in "Newport Tower" (1942). A very interesting development of the Viking hypothesis can be found in "The Lost Discovery" by Frederick J. Pohl (1952). "America 1355 - 1364" (1946), by Hjalmar R. Holand, develops this hypothesis for a particular period.

The actual recorded history of the tower is remarkably slight. Between 1629 and 1633 William Wood was in America. Returning to England, he published a map of eastern New England in 1634. At the present site of Newport he wrote, "Old Plymouth." Presumably ruins of an earlier settlement were visible there. It may be a reference to the existence of the stone tower. Now Sir Edmund Plowden petitioned for a patent to cover all of Long Island and part of the mainland in which mention is made of a "rownd stone towre" some distance from Long Island (east end) by sea. No one has suggested the existence of more than one tower, and most have not suggested it, so it would appear the Newport Tower was then standing and could be used. The Plowden petition for the New Albion patent was acted upon about 1632 by the king. When William Coddington led the colonists to Newport in 1639, it is stated that he saw the tower and endeavored to learn from the Indians their traditions concerning it. They had none. Our authority for this is B. J. Lossing, who, circa 1848, talked with Governor William C. Gibbs, then its owner, who passed on to him the tales handed down from the days of Coddington.

Assuming the probity of Lossing and of Gibbs, is it reasonable to suppose that a reliable story could be handed down for nearly 175 years? The writer must invoke the history of his own family to show that it is possible. His mother, then not over twelve, listened to the stories of the Revolution as told by Samuel Whittemore, her grandfather, concerning the acts of his great-great-great-grandfather on April 19, 1775. They related that he had chased the red-coats down a lane and killed them. The writer was told about these stories by his mother. She had thought these tales merely the tall stories a beloved grandfather liked to invent, so she had never learned them well enough to recount them in their entirety. It is now 200 yr since the events mentioned, just as the stories heard by Gibbs would have been nearly 209 yr old, at most, when repeated to Lossing. If you should visit Arlington, Mass., and go a block east from the square to a school grounds, you would find a stone reading substantially as follows: "Near this spot, on the 19th of April, 1775, Samuel Whittemore, then eighty years old, shot and killed three British soldiers. He was shot, beaten, bayoneted and left for dead. He recovered and lived to be ninety six years old." [illegible] would have been a little difficult for the relater of the stories to make them [illegible]y taller than the historical facts. At least the historians placing the monument are being given credit for reliability, in spite of many known instances in which historical societies have erred in the placing of monuments. A minor example of error was noted by George A. Richardson, the writer's brother. In checking the course of the Sullivan Trail, he found the road wound through the valleys of Easton, Pa., avoiding the hill now crowned by Lafayette College. The data comprised the pacings of distance by the military engineer of the expedition. This did not keep the local historical society from placing a road marker near the top of the hill, however. We must conclude that the story, retold by Lossing, placing the tower in existence as early as 1639 and sufficiently early to enable the Indians to forget all about its origin, cannot be ignored.

The first definite mentions of the tower occur in 1677. The more important of these is found in the will of Governor Benedict Arnold, bequeathing—"my Stone Built Wind-Mill"—. Since there is a record dating from August, 1675, to the effect that the windmill built and operated by Peter Easton had been destroyed by a storm, there had arisen the sudden public need for replacement. Apparently this was furnished, at least through the conversion of the Old Stone Tower, by Arnold. It is to be presumed from this statement that the windmill belonged to Arnold and that at least part of it was of stone. If the writer were to bequeath his home, he might write —"my frame-, and brick-built house"—. Nothing more would be implied (although his wife might well question the —"my"—).

There is remarkably little in the records concerning the tower during the years which followed, when practically no one denies that the tower was there. Yet lack of mention of the tower earlier is used seriously as an argument by historians to prove not only that the tower was not there but that Arnold built it. There is ample material extant to prove that the writer did not build his present home. In fact, it was started about 1856 and finished about 1870 or later. It is probable that the writer, though he may have seen this house many times during 33 yr, did not mention it earlier than 5 yr ago. It was purchased about 3 yr ago. Let it be supposed that a hundred years pass during which heirs, rag men, fire, war and the like produce such an attrition in written records that only a will such as that suggested should remain. Conditions become comparable with those relating to tower records. If the same method of argument should be used in this case as certain historians have used in that of the tower, it would then follow that the present writer's house must have been built by the writer, in spite of all inherent characteristics showing that it must have been built earlier.

At this point it seems well to consider a few of the many reasons why no previous mention of the tower has been found. The remark relative to heirs, rag men, fires, etc., obviously must be listed. A brief reference to "a stone ruin" would not keep a letter from being destroyed when the necessity of getting rid of old papers arose. In the year 1639, it may be assumed that many of the colonists were illiterate; hence the frequency of writing would not be great. Even partial literacy does not encourage long letters, or those which deal with anything excepting matters of importance to the writer. Besides, colonists as a class are a people engaged in a continual battle to maintain a marginal subsistence. This means that they must work as hard as they can to keep alive and must make some slight provisions for better times. Under such conditions even the literate people will be limited, in their written notes, to the more pressing matters.

Consider the behavior of a group of colonists arriving in a new land. Their leader will tend to take such notes of odd and unusual features as may deserve mention, while the other members will tend to disregard everything not directly related to securing a desirable home and lands. Hearsay testimony is available that William Coddington did note the tower and made a few relevant inquiries, although he may not have felt that it was worth recording. Aside from their natural concern with the pressing business of keeping alive, most of the colonists came from a country where ruins of all sorts, from a hump in the ground to a ruined castle or cathedral, were normal parts of the landscape. Being accustomed to the large and striking ruins, a small tower would be quite unimpressive. Besides, these same colonists might argue, men had been crossing the Atlantic for nearly 150 yr, so why be surprised if some of them built a tower. They knew full well that before the Pilgrims reached America the boats of the fishermen in ever increasing numbers were going west to the coast of Maine each year. They had been surprised to have Squanto speak to them in an English learned from the Popham colonists of Maine, but once that surprise wore off, they expected to see the signs of people who had come earlier.

There is still another consideration. Suppose the men and boys, noting the tower, thought of the freebooters of the Spanish Main, and conceived the idea that some of them had established a strong point here and might have hidden treasure. This tower now stood on the land of one of their number. In order to dig for treasure, an agreement would have to be drawn, granting the owner a portion of anything uncovered. In any case the owner would insist that the property should be returned to its original condition after digging. As farmers, the colonists would be required to keep the top soil separate from the hardpan below. So they would do a very thorough job of digging up the site and would find nothing. The normal artifacts of construction are scattered, any old metal is saved for such use as might be found, and the new artifacts of the diggers get into the back-fill. Rocks are loosened and removed from below columns to search for secret hiding places. Considering the work on their own properties that must have remained undone during this community dig, it seems certain that the good ladies had much to say to their men folk that would not be considered complimentary. Men are not noted for publicizing their errors and follies. The ladies, granting them literacy, do not care to publicize the shortcomings of their mates, for that might reflect upon them personally. The tower would be a most unpopular subject, so, the less said about it the better. By the time leisure to study the structure became available, most common knowledge would be gone.

Much has been made of the failure to find a stone tower as an asset in the listings for taxes. Suppose a man finds his land encumbered with a stone tower for which he can find no use, one too big to tear down, yet taking up valuable space. Human nature does not call such white elephants to the attention of the tax gatherer so that one may lose even more through the paying of taxes on the useless property. It would have been surprising if the tower had been mentioned in such lists before 1675. Even a governor must have decent respect for the interests of the colonists in drawing up such lists. Of course the owner is inclined to brag a bit when his own ability to recognize and seize an opportunity enables him to put his white elephant into a highly profitable business and at the same time gain the plaudits of his neighbors for his care for the public weal. He can now afford to pay the taxes. If mention of a tower in a letter to England might hurt a neighbor by getting such a structure on the tax lists, there would be good reason for not putting anything concerning the tower in letters written to England or elsewhere.

It would not be very difficult to frame other reasons why the tower would not be mentioned. Colonists, by and large, may tend to lack curiosity; business is so pressing there is no time for the day-dreamer, the poet, the artist or the student of anthropology. Kill the Indians, kill the animals; why waste time in learning about them? Before such things can receive proper attention, there must be leisure.

At least one archaeologist has investigated the ground inside the Newport Tower. During 1948 and 1949, William S. Godfrey was commissioned to do the work. Mr. Godfrey hoped to find Viking relics. The report of these investigations contains some interesting points. Under one of the columns there was quite a cavity that no competent engineer or builder would leave. It suggested treasure hunting at some past period. There were no signs of any Viking artifacts; in fact, strange as it might seem, the usual chipped stones and other debris of a construction site were missing. Godfrey classified the site as "undisturbed"; in other words, after the time of the builder, no one had dug there. But this finding was based on lack of admixture of top soil and hardpan below. Such artifacts as were found were those typical of the 17th century colonists. Coins dating from as early as 1696 were found. According to the usual mode of interpretation of data, the lack of normal construction debris should have indicated that the tower had never been built. This is a bit difficult to admit as the tower is certainly there. But according to the coins, the tower could not have been built before the first years of the 18th century; 1696 is nearly 20 yr later than 1675. If reference is made to the present writer's hypothesis that the site was dug up in the search for treasure, it will be seen that many of the oddities of the findings made by Mr. Godfrey would be normal to such a "disturbed" site. The present writer finds the "certainties" of Mr. Godfrey unwarranted by the findings as reported.

The purpose of the writer in these attacks upon the validity of the conclusions of others is to maintain open hypotheses regarding the origin which would otherwise be unavailable for use. The writer does not feel that he is unduly unfair to those who have worked in this field. One noted anthropologist was told that the small openings on the second floor of the tower were for signal-receiving purposes. His reply amounted to saying that this was certainly an unwarranted point of view. Obviously if a person wished to see, he would make the windows as large as possible so as to see better. This seemed to the writer rather an odd conclusion from an anthropologist, for such a person would know that the pupil of the human eye is only about 2/5 in. in diameter in the dark, so that a suitable hole at least that big would insure perfect vision. Besides, and this point was completely ignored, the viewer might find that enemy missiles were more dangerous than certain sight limitations. Besides it is possible the builder had no glass for such things. In that case, and in winter, the smaller the windows the better, provided they would serve their purpose otherwise. In view of such criticisms, showing a lack of what we like to consider "common sense," it becomes necessary to emphasize the writer's point of view.

If there is reason to suppose that the colonists of the 17th century did not build the tower but rather some one much earlier, the range of hypothesizing broadens. Frederick Pohl lists distant voyages on the Atlantic from early times until 1492 which suggest that many peoples have records of trips. Few of these indicate a stop long enough to permit the building of a tower. Only Holand offers a period of about nine years, which would be sufficient for such an undertaking.

Why should the present writer be so anxious to keep a wide range of hypotheses open? Although he considers the Arnold hypothesis unlikely, it has not been rejected; the only real rejections are of certain highly illogical conclusions drawn from such data as are available. The history of science is replete with the shifts back and forth between one reasonable hypothesis and another. To be sure slight changes may occur in "the same" hypothesis between the time it becomes ruling and the next, and such changes must be expected as knowledge increases. The failures of science have tended to occur when a reasonable hypothesis has been "proven" untenable by some mockery of logical thought. Those interested in securing a better understanding might well read, "In Search of Adam," by Herbert Wendt, a book dealing with the historical shifts in hypothesis, the great reigning minds refusing to consider objections, and persecuting those who held new points of view in conflict with "orthodoxy." It is not a pretty picture of the scientific mind in operation, but it is an excellent one of human beings swayed by petty emotions. A particular example of oscillations between hypotheses is related to spontaneous generation. Years ago, every one "knew" that maggots and germs developed spontaneously in putrid matter. Then Louis Pasteur came along and "proved" that spontaneous generation is impossible, thereby believing he had proven the fixity of species. Much later Harold Urey came along, performed his famous experiment using electric discharges in an atmosphere of ammonia, carbon dioxide and water vapor in the presence of water and was able to show that a wide range of organic compounds were formed thereby; hence the motion of spontaneous generation of life became a distinct probability. The regeneration of the earth's atmosphere to one primarily of nitrogen and oxygen by plant life could be shown to be probable. This is but one of a large number of examples.

BUILDING OF THE TOWER

Those who mapped the tower noted that the north and south columns had their centers in the meridian. It becomes of interest to see how this might be done. Most writers say that the north star, Polaris, also known as α-alpha Ursae Minoris would serve. They do not note that, though the distance of this star from the pole of the heavens is only 56' of arc this year, it was about 2°31' away in 1675 and nearly 4°43' in 1358 (600 yr ago). In no case is a single direct reading possible unless the observer has a modern transit instrument, a good chronometer properly set, and a copy of The American Ephemeris and Nautical Almanac. A simple method consists in taking numerous sights on this star during some 18 hr of a long winter night so as to determine the horizontal angle to it when furthest west (west elongation) and when furthest east (east elongation). Given the angle between these two extreme sights, that angle may be bisected and the line of bisection lies in the meridian. A somewhat shorter method might be used today to locate the meridian: as the stars change position with time, it might not apply in 1675 or 1358. δ Cassiopeiae had a right ascension of 1 hr 20' 53.69", and ζ Ursae Majoris had a right ascension 20'54.62" in 1925. A straight line between them would pass very close to the pole of the heavens. If this straight line were vertical at the instant of observation, it would determine the meridian within a few seconds of arc. For more precise work, this line when vertical would serve as a time origin from which to measure about 13' of time until Polaris were at upper or lower culmination and on the meridian. The time for observation could be shortened very materially if such a choice were available to the observer. It would be unreasonable to suppose the quicker method would be known to the builder, hence the long method would be forced upon him with all its difficulties and discomforts.

If the builder had knowledge of the making and placing of sun-dials, a much more comfortable method would be available. The sun dial depends upon the sun itself and is most conveniently used about June 1st, although its precision might be adequate for the purpose at any time. First set up a table with a top as level as possible. Near the south edge of the top and about the center of the width make a point on the surface. With this point as a center, scribe several circular arcs between table edges. Support a plumb line so that the line intersects the table top at this center point. On this line and a suitable distance above the table place a knot or very small bead. The equipment is ready. On a bright sunny day, as soon as the shadow of the bead is on the table, start marking its position with care at frequent intervals. Do this until the shadow of the bead leaves the table. Now draw a smooth curve accurately through the points so marked. This curve will cross each arc in two points. The chord of any arc between the points of crossing may be bisected. These chordal bisections and the center point of the arcs will all lie on the meridian of the place within about 1' or 2' of arc.

To a builder who will probably start his job in the spring, the last method will be preferred. Besides, as will be seen later, such a table top can serve many purposes. If established at the center of the tower site, the tower can be laid down to scale on this surface and necessary building lines can be taken off directly. Furthermore, it forms an excellent surface upon which to lay out the window openings. But first the bearings of various landmarks must be obtained and perhaps the elevations of some will be required.

For the establishment of bearings, a simple instrument can be set up very easily. First, prepare very carefully a straight edge. This may be 3 ft long. At one end, erect a thin sheet of wood with a tiny eye aperture. It would be desirable to have this opening exactly over the edge of the straight edge and at least 3 in. above its top. At the other end of the rule and preferably in the vertical plane through the working edge, establish a truly vertical line comprising a thread. Once established, it can be fastened to a suitable frame giving a clear opening about 4 in. or 5 in. high. The alidade is now ready for use. Line up the straight edge on the tower center of the plan. Sight on the desired landmark. When both tower center and the landmark are located, the bearing line for the landmark can be drawn. With a peep hole of the order of 1/16 in. in diameter and a thread of the order of 0.01 in. in diameter, bearings good to about 1' of arc may be taken. All necessary bearing lines can be set down on the table top plan.

Vertical angles may be required to certain points. The alidade could be adapted to such very easily. A movable frame with a perfectly horizontal thread is required crossing the vertical thread at that end. A level of any sort (a trough with water in it and floating wood blocks with pins) can be used to establish the point at which the horizontal thread is exactly at the height of the peep hole. By moving the thread up and down, and measuring the movement, the dip angle may be determined. In this way the builder could determine the dip to water level about 1600 ft away. Military pacing of the slope, carried out with care, plus this angle, would yield the table-top elevation within about 1 ft.

It is easy to show that vertical angles could be determined about as well as horizontal angles, and with about the same precision. The accuracy of the device is adequate to measure the tip to the horizon. The tower horizon distance is of the order of 11-1/2 miles. The horizon dip is about 5/6 of 1' per mile. The actual dip is reduced by about 1/7 by refraction. Hence the actual dip of about 9-1/2' would be reduced to about 8' of arc. If these facts were not known to the builder, he might set up his alidade for vertical angles by sighting on the horizon line on a very clear day, and so have his dips nearly 8' too high. It is not obvious that this can be proven through the tower itself. It is not even obvious that the builder would feel the need of such equipment for vertical angles. The considerable precision with which the tower windows were located suggests some such apparatus was used in laying out the tower, or at least might have been used. Of course a careful surveyor would realize that to adjust his level to the horizontal, he must reverse his instrument and sight of the same horizon. If he did so, he would find a change of 16' in the horizon location.

All that has been shown so far is that an engineer could easily acquire the required instruments for finding the meridian and for the accurate laying out of the tower and its windows from a plan established on a table top. By 1358 engineering ability of a high order could be expected of any man who had been trained in church construction. At that time the best of the Gothic cathedrals were being built. Just as in the case of the establishment of the base for the columns of this tower a circular trench was dug to a suitable depth and filled with rock, so in the case of the Gothic cathedrals and other structures from about 1100 A.D. onwards trenches of this sort were dug and filled with rock for the support of walls and the like. It is probable that any time after the Crusades there were men who could construct and locate sun dials. It seems reasonably certain that such knowledge was available by 1300 A.D. Certainly the means for establishing the meridian in the orientation of churches were known throughout the period of Gothic cathedral construction.

DESIGN OF THE TOWER

Before considering the layout of the windows, it seems well to consider the actual design of the tower. For this purpose it is necessary to set down the dimensions which will be used, for the published dimensions are not always in agreement. In general these will be taken from the drawings of Rowe, except for timbers, and those may be taken from the measurements of Mason. Where there is a question relative to the exact dimension to use, the writer has chosen the nearest measure in Norse feet and inches, after the suggestion of Pohl. A Norse foot is said to have been 12.3543 English in. or 1.02953 English ft. Both dimensions have been given in Table 1.

In checking the tower design, weights of stone work will be based on 150 lb per cu. ft. The probable range is from 146 to 153. This value applies to rubble, including some granite, in lime mortar. It will be assumed that the wood is white oak weighing 46 lb per cu. ft. For such wood, the allowable stresses might be about as follows (lacking better information, present-day values will be used): For white oak, the extreme fiber stress in bending may be 1100 lb psi. If shear is of consequence, the allowable value may be taken as 100 psi with the fiber. Bearing across the grain may be taken as 800 psi.

It is of interest to know that allowable compression on such rubble masonry ranges from about 50 to 83 psi with the majority of specifications setting 70 psi. The allowable bearing pressure on hard clay ranges from 3 to 4 tons (short) per sq. ft. These values have been taken from the digests of building codes for United States cities before 1919 as given in Cambria Steel Handbook.

An oak beam 12 in. by 12 in., 18 ft long, uniformly loaded, can support 11,730 lb. A foot width of 2 in. plank supported on 66 in. clear span will carry 194 psf of floor. There are, effectively, 269.7 sq ft of floor. Two beams can carry about 87 psf including the weight of the floor. The weight of the floor is about 13.8 psf. Hence the live loading is about 73 psf. The total load, floor and live load, amounts to about 23,460 lb, the amount the two beams can carry. The live load corresponds, roughly, to what might be considered desirable for offices on upper floors, or a somewhat low value for places of public assembly. From this analysis, the floors could have been designed for the type of occupancy that might be expected of them. Let the assumed floor and occupancy weight be set at 24,000 lb each, first and second floors.

TABLE 1.—TOWER DIMENSIONS

Item	Description	English Feet	Norse Feet
1	Outer Diameter of Tower	23.16	22' 6"
2	Inner Diameter of Tower	18.53	18' 0"
3	Column Base Diameter	4.29	4' 2"
4	Column Shaft Diameter	3.26	3' 2"
5	Column Bases 3,4,5,7 Datum Above $\frac{1}{2}'$ above ground		
6	Ground Height About 84 feet above Mean Sea Level		
7	To top First Floor from Datum	12.01	11' 8"
8	To top Second Floor from Datum	19.56'	19' 0"
9	To top Present Wall from Datum	25.31	
10	Est. Height to Top of Wall above Datum	27.11	26' 4"
11	Bottom First Floor Beams to Top Floor	3.09	3' 0"
12	Second Floor Pads to Top Second Floor	1.20	1' 2"
13	Height to Center of Arches	9.95	9' 8"
14	Estimated Mean Height of Arches	9.40	
15	Column Height to Top Outer Pads from Datum	8.58	8' 4"
16	Column Height to Top First Floor Pads	8.92	8' 8"
17	Height to C.L. First Floor NE Window	16.73	16' 3"
18	do. SE Window	15.44	15' 0"
19	do. W Window	13.91	13' 6"
20	do. N Port	16.13	15' 8"
21	Height to C.L. Second Floor NE Port	21.79	21' 2"
22	do. SE Port	24.97	24' 3"
23	do. W Port	24.19	23' 6"
24	Fireplace Height to C.L. Arch	4.80	4' 8"
25	Height Datum to C.L. Flue Holes	23.25	22' 7"
26	Square Opening between First Floor Beams	5.32	5' 2"
27	Floor Thicknesses 2"		
28	Estimated Size of Floor Beams 12 x 12 inches.		
29	Clear Spacing of Second Floor Beams	5.15	5' 0"

It is reasonable to suppose that the roof might have been conical with a 45° apex angle. Including roofing, the weight might be taken as 22 psf on 270 sq ft on 270 sq ft or, say, 6000 lb.

It is of interest to note that the segmental area cut out of the column under the outer pads is between 5 and 6 in. high at the center of the segment. The central angle subtended lies between 85° and 94°. It seems well to assume 90° The ratio of height to radius is 0.2929, the ratio of chord to radius is 1.4142, the ratio of area to the square of the radius is 0.2854. The center of gravity of the segment is approximately 40% of the height above the base. The total column cross section is 1202 sq in. The segment area is 109.2 sq in. The center of gravity of the segment is 16.12 in. from the column center. The remaining area has a center of gravity distance 1.61 in. from the column center. The height of the segment is 5.73 in., the chord thereof is 27.66 in. To have the total loading on the shaft of the column centered, the loading on the outer caps must be 10% (1.61/16.12) of the total load of the tower thereabove. This, of course, assumes that the builder knew how to calculate moments, which it now seems reasonable to do.

The weight of the stone in the tower, neglecting openings and changes in section, from the mean height of the arches to the top of the tower amounts to 402,700 lb on the basis of the dimensions given. Add to this the loads due to two floors and the roof, or 54,000 lb. The total dead and live load becomes 456,700 lb, neglecting any wind or ice loads on the roof. It seems reasonable to assume the total does not exceed 460,000 lb. But if this is true, then the outer pads of the columns were intended to support 10% thereof, or 46,000 lb. It is obvious that each of these pads could take two 12 in. by 12 in. beams. Assuming a snow or other live load of 40 psf and a dead load of about 12 psf, it becomes possible to calculate the size of the circular enclosure one story high. Suppose that 42% of the load is carried on the inner supports, then the total load is 110,000 lb. The diameter of the enclosure becomes about 56.8 ft. It is not unreasonable to believe that the outside diameter of the enclosing wall was intended to be 60 Norse ft, allowing approximately 1 ft 6 in. for wall thickness and pilasters on the inner sides of the wall at beam supports.

It will be noted that the roof beams are about 16 ft long in the clear. Using sixteen such beams, the spacing at the outer wall is about 11.8 ft, or about 11 ft in the clear. The loading for 2 in. planking under these conditions is about 50 psf. It can be shown that a 12 in. by 12 in. beam 200 in. in length with the loading imposed will be safe. The required moment is about 279,000 in.-lb while that available or allowable is 317,000 in.-lb. The weight of such a structure, neglecting roofing, is about 12.2 psf.

It becomes of interest to see what stresses were allowed in the columns. Now the weight of the columns is about 94,200 lb, so the total weight on the shafts amounts to 600,000 lb, including also the loading on the outer pads of the columns. As the cross-sectional area of the eight columns totals 9616 sq. in., the stress amounts to about 62 psi. The code range in 1919 was from 50 to 90 psi with the majority about 70 psi. There is considerable evidence in this design that the builder was well acquainted with rules for loading his masonry in building a permanent structure.

Now the areas of the eight column bases amounts to 115.6 sq. ft. The loading per square foot applies to the foundation is about 2.6 short tons per sq. ft. From 2 to 5 tons per sq ft would be permitted on hard clay, when the ruling intensity is about 3 tons per sq ft.

This tower appears to have been built and designed by a person well acquainted with stresses to appropriate design requirements and loads suitable for the occupancies contemplated. There is reason to believe that this man might have been trained in the construction of churches, a thing far from impossible at about 1360 A.D. This was not simply a tower built so solidly that practically nothing could cast it down: it was a tower designed to do a specific job and to use the minimum of material for a permanent structure. The designer knew how to balance his eccentric loadings so that the column shafts would be centrally loaded. The size of structure derivable from the assumption that this was done yields a structure of pleasing appearance and of considerable size (Fig. 2). Others have suggested that stone arches might have been used for the support of the roof of the outside structure. This seems highly improbable, for the size of arches required would have produced a roof which would have hidden the windows of the first floor. Using beams as suggested, the roof, either flat or with a slight slope, would have been sufficiently high to keep the stonework of the arches within the first floor of the whole structure, yet would have covered the arches completely from the outside without infringing upon the second floor windows. A suitable outer wall might well have been given a high, perhaps crenelated parapet so that the defenders might protect the roof from direct access by an enemy and safeguard the windows from serving as means of ingress to the tower. Now it is possible, as Holand has suggested, that the west window of the first floor was intended as a temporary ingress into the tower. But with the outer section roofed, this would have been a very clumsy means of entrance. The first floor differs from the second in having a space between the timbers nearly 5 ft 4 in. sq. As there was no stairway to this floor, it seems obvious that a trap door was

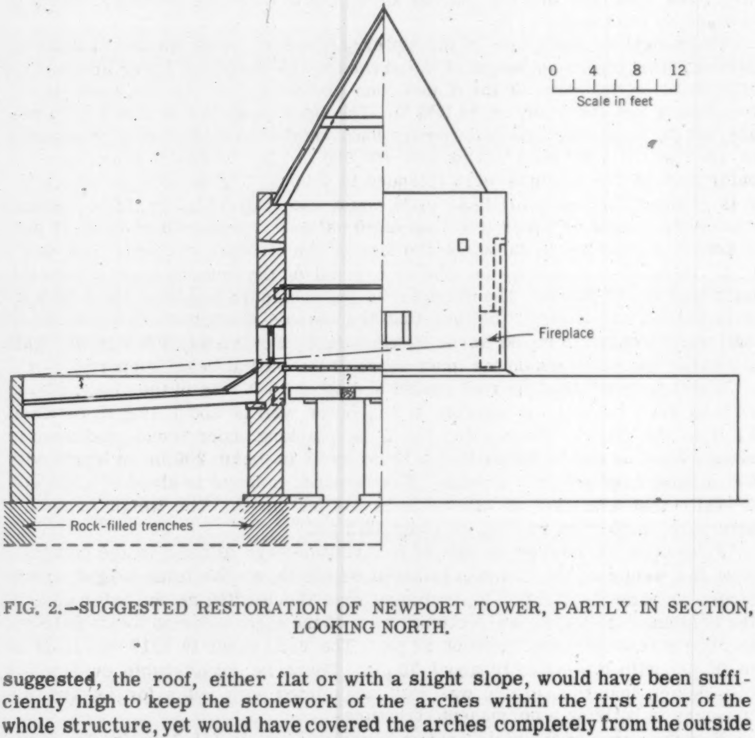

FIG. 2.—SUGGESTED RESTORATION OF NEWPORT TOWER, PARTLY IN SECTION, LOOKING NORTH.

to be used in this space permitting protected access to that first floor when the enclosure was completed.

In the case of the first floor roof outside the tower and of the tower roof it seems that the roof planking was protected. The tower, with ports from the fireplace close below the roof, could not have an overhanging roof edge which would be in danger of catching fire; that edge had to be protected. The flue holes were only about 7 ft above the top of the fireplace. So, with a good fire on the hearth, flames, sparks and hot gases might be spewed out.

If this indicates that the builder of the tower was a competent engineer and builder, then it becomes obvious that he would have designed and built a structure suited to his clients. Furthermore, if this engineer knew better designs for fireplaces, it seems reasonable to believe he would have used one in a building showing the character of this stone structure. As he used a fireplace commonly accepted in or before 1400 A.D., but very inferior in design and concept in 1675 A.D., it seems necessary to place the construction of the tower near the earlier date. From used to be made of the windows, it would seem essential that he employ the best fireplace obtainable. Certainly nothing about the tower suggests that the client ever instructed the designer that it was to be constructed for flour-milling. The floors are not of the right strength; a fireplace is placed disadvantageously; there are no adequate means for getting machinery into the structure; there is no reason for a surrounding one story structure which might interfere with the use of a mill wheel; and no protection for the flour milling operation is available, and no adequate means for getting meal in and flour out. But if the time came when the tower was needed as a mill, certainly a reasonable adaptation could have been made at a cost, slight, when compared with that of building an entirely new supporting tower.

SIGNALLING

It is now advisable to follow the layout of the structure for signalling purposes. In this case it is necessary to proceed from the final layout to the means used for producing it. The means for setting down the essential data have been suggested.

Many suggested that the windows might have been used for watch tower or signalling purposes. Aside from this article, there appears to have been no published study investigating the nature of such watch and signal means or the significance thereof.

The first step in the investigation comprises determining from cross-sections of the tower through the window openings on the two floors the extreme bearing lines for all openings. Confining attention to the second floor first, since its layout is somewhat simpler, and, in fact, suggests a signal-receiving function. Fig. 3 shows the results of such measurements. Window W5 to the northeast has a seeing range from an azimuth of 7° to 59°. Window W6 to the southeast has a seeing range from an azimuth of 112-1/2° to 162-1/2°. Window W7 to the west has a seeing range from an azimuth of 248° to 285-1/2°. The ports are not big enough for a man to stick his head out, so his range is limited by the cut-off of view as he passes before the opening being used. Of course this arrangement has a distinct advantage in locating the sending signal, particularly if it is a fire at night. Such a station could be accurately plotted on the far wall, partly by the ray at horizontal cut-off of the

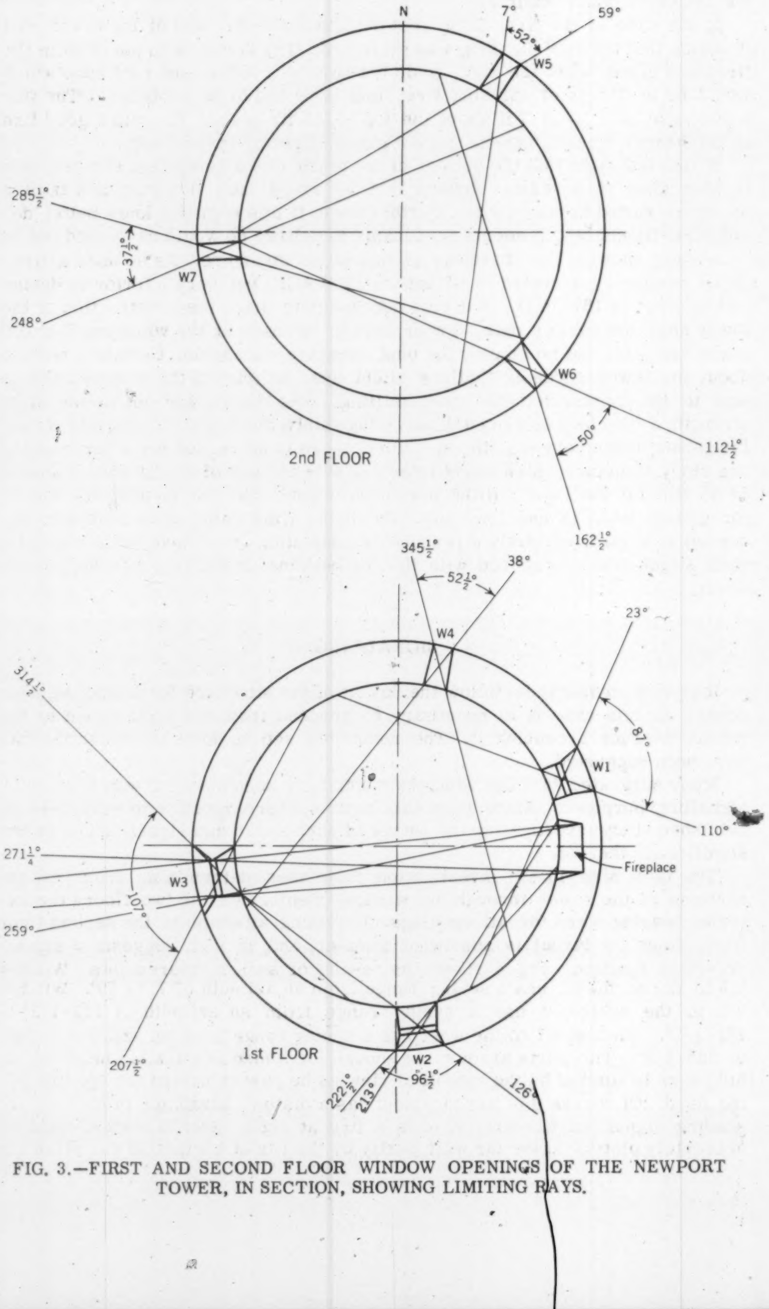
N
7°
59°
52°
W5
285½°
37½°
W7
248°
W6
50°
112½°
2nd FLOOR
162½°
345½°
38°
52½°
23°
W4
314½°
87°
W1
110°
Fireplace
271¼°
W3
107°
259°
1st FLOOR
207½°
W2
96½°
222½°
213°
126°

FIG. 3.—FIRST AND SECOND FLOOR WINDOW OPENINGS OF THE NEWPORT TOWER, IN SECTION, SHOWING LIMITING RAYS.

signal, partly by the ray of vertical cut-off by the horizontal edge of the window. If only horizontal cut-off is used, a line on the far wall would identify it. If both cut-offs, a point on the far wall would suffice.

The next step in the study is to lay down these bearings from the tower center upon a chart of the region in order to determine what points might have been of interest to the builder. From such a chart we determine the location of the tower as, approximately, 41°29' 9'' N. Lat., 71°18'38'' W. Long. In part the Newport, R. I. U. S. Geological Survey Map of 1944, reprinted 1950 was used, in part the U. S. Coast and Geodetic Survey Chart No. 115, Cuttyhunk to th Block Island, reissued August, 1916. As of 1939, the magnetic declination was given as 14-1/2° west of true north while in 1915 is was 13°10' west of true north. In 1915 the declination was increasing at the rate of 6' per yr. Obviously the rate was not maintained. The figure of 15.6° (15°36') shown on Fig. 4 is merely an approximation.

Laying down the bearings so determined upon the Geodetic Chart No. 115, a figure, showing the arrangement of the second floor rays is obtained. From window W5 the whole width of the land approach along Aquidneck Island is covered. There are four possible high spots for signalling in this region visible from the tower. The first is J, or Bliss Hill, El. 148 ft, about 1.5 miles to the north, with a horizon distance of 13 nautical miles. The second, slightly to the west, is K, apparently the point determining the extreme western ray for this window. Station K controls all water approaches from the north and conditions on Conanicut (G) and Prudence (H) Islands. This station is known as Middletown Hill, it is at El. 164 ft, is 3.6 miles from the tower, and has a horizon distance of 13.6 miles (all miles are nautical). Now this station might serve to relay messages from L, or Turkey Hill, at El. 275 ft and 5.9 miles from the tower. Its horizon distance is 17.6 miles. It is more probable that direct signals were used from L. Now L, M (Slate Hill, El. 265 ft, 3.8 miles from the tower, horizon distance 17.6 miles), N (Howland School Hill, El. 183 ft, 3.2 miles from the tower, horizon distance 14.4 miles) determine the outlines of a ridge blocking the view from points beyond. With a few small outposts, the land approaches to the tower were well protected and danger could be signalled quickly from any controlling high point. Obviously any one of these points might be worth study from the standpoint of locating possible artifacts. The number to be expected is small, of course. Although the eastern limit of window W5 is a bit uncertain, certainly the whole range down to station O (Paradise Rocks, El. 170 ft, 2.3 miles from the tower, 13.9 miles horizon distance) is covered. Station I (Tonomy Hill, El. 150 ft, 1.5 miles distant from the tower, 13.0 miles horizon distance) appears to be outside the range of observation. This is not important as the neighboring point J is within sight.

Window W6 has its extreme northern ray fully determined by BB and CC (West and East Islands south of Sakonnet Point, El. 22 ft, about 5.6 miles from the tower, with 5 miles horizon distance). No ship would approach north of this line because of shoal water. It is of interest to note that HH (Gay Head, Martha's Vineyard, El. 147 ft, 22.9 miles from the tower, 12.9 miles horizon distance) is within the zone of visibility. As the horizon distance of the tower (El. 109 ft) is 11.1 miles, a signal on this hill would be seen as a fire just above the horizon. The distance being 22.9 miles, the sum of the horizon distances of 11.1 plus 12.9 = 24 miles, which is a greater distance and thus insures visibility. Perhaps a better beacon might be H'H' (also Gay Head, Martha's Vineyard, El. 170 ft, 24 miles from the tower, horizon distance 13.9 miles).

A few words might be said at this point concerning "horizon distance" and visibility. Due to the curvature of the earth, a straight line tangent to the surface of the sea at "the horizon" will intersect an elevation at a height proportional to the square of the distance. If the height h is in feet above sea-level, and the distance to the horizon is D_s (statue miles), $D_s = 1.224\sqrt{h}$. If D_n is for nautical miles of 6080.2 ft), $D_n = 1.0663\sqrt{h}$. Actually the refraction of the air causes the light to bend around the earth so that a body which should appear to have its top in the horizon will be seen with considerable elevation above the horizon. Correction can be made to the tabular horizon distances by adding about 8% thereto. The actual refraction varies slightly. Now the straight line from an elevation to the horizon dips 60'' of arc for each nautical mile the horizon is distant. Due to refraction, however, the apparent dip is reduced to about 51'' per nautical mile.

In looking between the high points, the test of mutual visibility is, in the limit, the light ray, between that grazes the horizon. Then the distance between the two elevations cannot exceed the sum of their respective horizon distances. Actual visibility requires that the horizon distances corrected for refraction should be used.

To continue the discussion of W6; the southern extreme ray has no apparent limitation except coverage of the southeast sea approaches fairly well towards the south of the tower. Perhaps more information will be gathered from the first-floor windows.

Window W7 has its southern extreme ray so located that it skims the 40 ft elevation ground west of Brenton Cove (site of Fort Adams) and reaches the East Passage channel about 9 ft above the water. It would appear that a ship on course might have its forecastle or poop some 10 ft to 12 ft above the waterline to be certain of being seen from the tower. The distance to such a ship would be 2.45 miles. At night rigging would be difficult to see, though a light on the poop could be picked up easily. The northern extreme ray is not so easy to locate, but it appears to have been set to mark the northern limit of Goat Island (W) in the early days before filling was done. There are two other possibilities, perhaps. There is a hill at C (Wakefield Hill, El. 166 ft, 7.9 miles from the tower, horizon distance 14.0 miles) from which signals could be received. It may have purpose later. A rise at D (Boston Neck, El. 95 ft, 6.3 miles from the tower, 10.4 miles horizon distance) does not seem important. Either E (Tower Hill, El. 184 ft, 6.8 miles from the tower, horizon distance 14.4 miles) or F (McSparran Hill, El. 237 ft, 6.6 miles from the tower, 16.4 miles horizon distance) might have significance. Of course such points as AA (Southwest Point, Conanicut Island, El. 70 ft, 3.0 miles to the tower, horizon distance 8.9 miles), V (The Dumplings, El. 25 ft, 2.1 miles from the tower, 5.3 miles horizon distance) and Goat Island W (El. O 15 ft, 0.8 miles from the tower) might be of possible interest as signalling points; but they are more important as points to be watched.

The above covers the layout of the signal-receiving second floor. The builder was greatly interested in three things: the land approaches and possible outposts and beacon on Aquidneck Island; the sea approaches from the southeast; and the channel windings to the west. He was not interested in the sea approaches from the southwest so far as this second-floor receiving center is concerned.

It is possible the southwest approaches were watched otherwise, as by an outpost on Block Island, at an azimuth of 214° from the tower at a distance of 18.8 miles to its Beacon Hill at El. 211 ft and 14.5 miles horizon distance. It is possible that C (Wakefield Hill) served as such an outpost. But it seems more probable that direct signalling from Block Island to the second floor of the tower would have been used if there were any reason to expect vessels to approach from the quarter. Since there are no such provisions, it must be considered quite probable that no vessels were expected. If so, then it is to be doubted that this tower was built at any time during the 17th century, for the Dutch were a military threat during most of that period and most certainly a watch would have been kept in that direction.

Suppose, however, that we consider the conditions of Holand's 1355 A.D. colonists or their expedition. In this case there were no ships to be expected from the west. Any ships sent out in search of the West Greenland colonists would have sailed east and would return from that direction. A most likely approach by such ships would be through Vineyard Sound north of Martha's Vineyard, bearing south of Cuttyhunk Island. Any supply ships might be expected to arrive either through this channel, or perhaps from the southeast, south of Martha's Vineyard. There would always be the possibility that enemy or pirate vessels might appear from that direction, particularly any that might follow a supply ship to a promising colony or small port. Nothing else could expected to arrive by sea. Certainly these considerations fit the provisions made by the tower builder much better than any others that might be suggested at present.

With these points in mind, attention can be given to the first floor. If the second floor was, as it appears to have been, a signal-receiving station, it might well be that the first floor was intended as a beacon, or as a signal-sending station. At night a bright fire on the hearth would provide the necessary light. Hence it seems desirable to consider not only the extreme rays through the first floor windows, but also the rays marking the first appearance of direct fire as well as the region in which constant fire light may be seen. Fig. 3, first floor shows the results of such studies, in determining the necessary azimuths of lines. Window W1 to the east has rays with azimuths of 23° and 110°, and direct firelight is not seen. For window W2 to the south, the extreme azimuths are 126°, and 222-1/2°. Now the beginning of direct firelight is at 213°. The west window, W3, has extreme azimuths of 207-1/2°, and 314-1/2°. The beginning or end of direct firelight are lines with azimuths of 259°, and 271-1/4°. Over an angle of about 1 in 20, or nearly 3° at the center of the range, direct firelight remains constant. Window W4 has no direct firelight. Its range is from 345-1/2° to 38°. With these azimuths, reference may be made to Fig. 4 where they are laid down on the chart.

Window W1 appears to be fully determined by bearings. Station L at Turkey Hill (El. 275 ft, 5.9 miles from the tower, horizon distance 17.6 miles) corresponds to the extreme north bearing, while Sakonnet Point Q (El. 45 ft (maximum), 5.5 miles from the tower, 7.1 miles horizon distance) sets the extreme south bearing. Once again the limits of shallow water appear. This time it appears that the northeast window is arranged so as not to be seen when in operation by any approaching ship. All stations such as L, M, N, O (Paradise Rocks, El. 170 ft, 2.3 miles from the tower, 13.9 miles horizon distance) S (Easton Point, El. 145 ft maximum, 1.6 miles from the tower, horizon distance 12.8 miles), R (Sachuest Point, El. 45 ft (maximum) 3.0 miles from

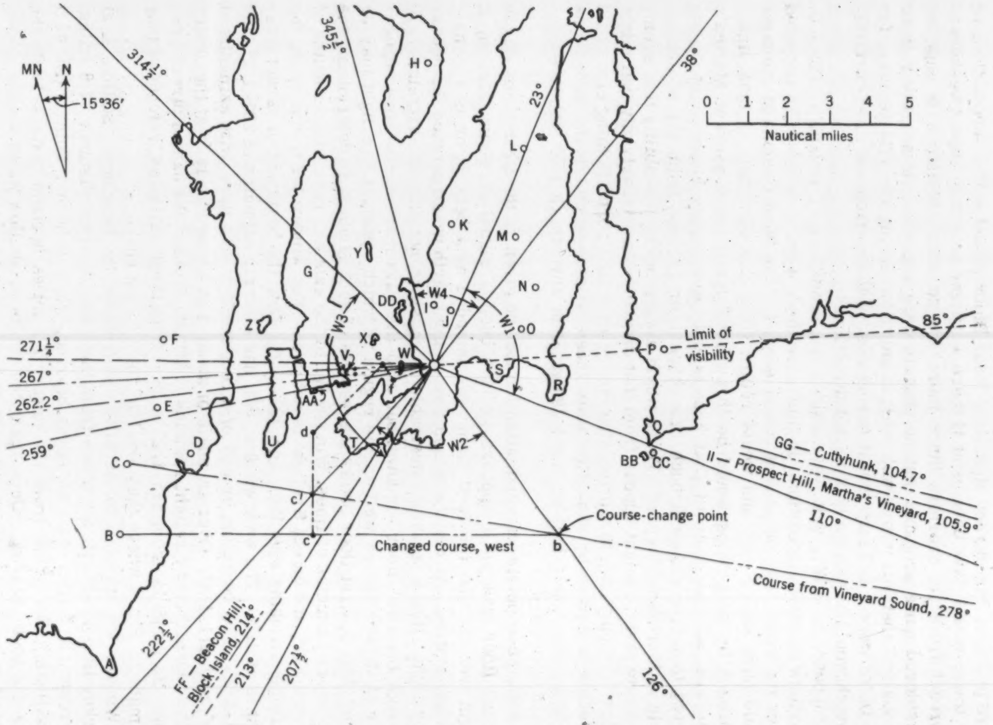
MN
N
15°36′
0 1 2 3 4 5
Nautical miles
314½°
345½°
23°
38°
85°
Limit of visibility
271¼°
267°
262.2°
259°
222½°
FF — Beacon Hill, Block Island, 214°
213°
207½°
126°
GG — Cuttyhunk, 104.7°
II — Prospect Hill, Martha's Vineyard, 105.9°
110°
Course-change point
Changed course, west
Course from Vineyard Sound, 278°
W1
W2
W3
W4
A
B
C
D
E
F
G
H
I
J
K
L
M
N
O
P
Q
R
S
T
U
V
W
X
Y
Z
AA
BB
CC
DD
b
c
c′
d
e

FIG. 4.—FIRST FLOOR WINDOWS OF NEWPORT TOWER SHOWING LIMITING RAYS AND PROPOSED SAILING COURSE.

the tower, 7.1 miles horizon distance) and P (Little Compton, El. 65 ft (maximum), 5.7 miles from the tower, 8.6 miles horizon distance) are within range of the signal. A line with an azimuth of 85° marks the most northerly points not cut off by the ridge L, M, N, O and S. Now it will be seen that Cuttyhunk Island, GG (El. 154 ft, 14.4 miles from the tower, 13.2 miles horizon distance) might see the light from the window, but it is improbable on account of distance. Similarly for Prospect Hill II (Martha's Vineyard, El. 308 ft, 26.3 miles from the tower, 18.6 miles horizon distance).

Departure a lies off the map for a vessel sailing from Vineyard Sound towards Newport. A suitable landmark on a desirable course is needed. Now Wakefield Hill at C (El. 166 ft, 7.9 miles to the tower, 14.0 mile horizon distance) seems like a good possibility. Hence the straight line point of departure a to C. Now a is 14.3 miles from the tower. It is 19.7 miles from Wakefield Hill. Suppose a man's eye aboard a ship is 16 ft above the water. The observer's horizon distance is 4.2 miles. Then, 4.2, plus 14.3 miles is 18.5 miles. Allowing for normal refraction add 8% and the hill top is on the horizon at 20.0 miles for the ship's observer. Hence Wakefield Hill is a very logical landmark, as it will "rise" above the horizon very shortly after the ship takes its departure from a. Hence the straight line as drawn seems like a logical course.

Now it will be seen that the extreme ray to the east from the south window, W2, cuts the line aC at b. This could be a course-change point. If so, it seems well to check for a new landmark. Two choices, one behind the other, are available. The first is B, a hill on the Point Judith peninsula (El. 97 ft, tower distance 6.7 miles, horizon distance 10.2 miles. The second is Broad Hill (not shown) (El. of 226 ft, with a horizon distance of 16.0 miles). Broad Hill is 3.74 miles west of B on Point Judith. The distance of course point b from hill B is 11.59 miles. The distance to Broad Hill is 15.33 miles. The observer on the ship would see Broad Hill at 15.6 miles, allowing for refraction, so the hill is visible. Similarly Broad Hill would be on the horizon at 20.2 miles, allowing for refraction. The two hills together may serve as a suitable beacon for a course due west from b.

It will be seen that if there is no change of course at point b, the ship will reach a point where the line of light from the west window is visible, then begins to see direct firelight through the south window, and the direct firelight disappears at c'. This would be the point at which the course should be changed to true north. It lacks a certain degree of safety, since the pilot may not be sure of the point of true cut-off, and thus turn north too soon. Following the course due west from b, a point c is reached, very probably characterized by maximum direct firelight intensity. This seems to be a much safer method for determining the turning point, so it seems very plausible that the intended course is actually a to b to c, then due north.

In passing it may be noted that the direct firelight from the tower does envelop Block Island. The Hill FF (Beacon Hill, El. 211 ft) is 18.8 miles from the tower and has a horizon distance of 14.5 miles. As the tower horizon distance is about 10.4 miles, the sum is 24.9 miles which is a great deal more than the actual distance, even without allowing for refraction. Incidentally, also, a straight line from the tower to FF passes directly through point c. It is doubtful if this would be a matter of coincidences. When available as an outpost, the tower and a beacon on Beacon Hill of Block Island might well be the normal manner of marking the turning point. But the builder appears to have provided for the contingency that the Block Island beacon would be missing, so made final guidance rest primarily on the tower signals alone.

As the pilot proceeds northwards between c and d, he may have occasional glimpses of the west window of the tower, W3. At about point d, he will be able to see the west window wall light over the Fort Adams headland. He could turn due northeast (true) at this point. If a beacon were lighted at I or J, he would head directly towards it, shortly coming in view of the second floor west window. Whether there is a beacon at I or J or not, the builder has provided that the ship can be brought into port. Continuing along the northeast course, he shortly crosses the south line of direct firelight, sees the brightness grow, then become constant, and finally grow less and vanish, leaving only the much fainter wall light. Just as the direct firelight disappears, the course must be changed to southeast (true). The ship recrosses the zones of direct firelight until once again the direct light vanishes, then makes a slow turn to port until the tower light lies directly ahead. Using the tower as a beacon, the ship proceeds straight to port.

This leaves the north window, W4, for consideration. In passing it might be noted that nearly full direct firelight can be seen from E to the west of Narragansett Bay, a place called Tower Hill (El. 184 ft, 6.8 miles from the tower, 14.4 miles horizon distance). This is a very logical point for a lookout, as signals can be received most easily, and return signals can be made from there. It is also about 16.4 miles from Beacon Hill (FF) on Block Island, so is in plain sight of that elevation. As the tower is only 18.8 miles from Block Island, it might seem that direct signalling, tower to Block Island, would be preferred to a roundabout course.

Window W4 to the north seems intended for short-distance signalling only. It is a small port. The beacon points I and J lie at a short distance. More remote locations are H (south end of Prudence Island, El. 175 ft, 7.5 miles from the tower, 14.1 miles horizon distance) also K, L and M. As L and M can receive from the east window, and as M can see both the port and the east window, it is possible this was intentional, and that M was the prime outpost in this direction. Point K is at approximately the same distance, but might have difficulty seeing the north port. The east window is not visible.

The foregoing sums up our knowledge of the window arrangements so far as horizontal angles are concerned. It becomes of some interest to determine the vertical angles to some of these points, as a check. Using window W5, all targets I through O reach the far wall of the tower from 0.55 in. to 1.48 in. below the bottom edge of the window. This allows for earth curvature, refraction, and the like. This window is low, because it can be looked through from the stairs. Otherwise it would be the highest window. The northeast window on the first floor stands about 0.6 ft higher at the center than the first floor north port. But the bottom edges are about the same height. They point to high ground. The southeast window, first floor, is about 1.53 ft (1.50 Norse ft) higher than the west window on center line, but is about 1.29 ft lower than the northeast window. The line of sight on the far wall is about 1.28 in higher than the window sill assuming 3.33 miles from the tower to point B on the course. This is about enough difference in window elevation to have the same portion of the fire visible through the south windows and the west windows. The south window is 2.86 ft high, the west window (clear) about 2.25 ft high. A difference in center height of 1.53 ft makes a difference of about 1.33 ft on the bottom edges of the window. Although not exact on the basis of such calculations, it is obvious that deflections upwards and downwards, as well as the portion of the fire to be exposed, all entered into the locating of the windows. As a result, this writer finds little left to chance. Every dimension received due consideration by the builder for the purpose for which the window was to serve. Just as not all dimensions are critical, but may be left to judgment, so in this case places which do not quite check unquestionably were known to be satisfactory even with modifications for some matter of convenience. A modern engineer makes sure the design is the minimum for doing the work, then makes such changes as facilitate production, or the like, or insure reasonable standardization. This is done without reducing the effectiveness of the device for its intended purpose.

There is one more test to be applied to the plan that has developed during the course of these studies. Is the light sufficient? Will any suggested uses be excluded by such a test? First a few principles must be enumerated. A light source of 1 cp is, in theory, a small, uniformly bright spherical surface, such that at the distance of 1 ft there is a degree of illumination known as 1 ft-c. As the spherical surface 1 ft from the center of the source has an area of 4π sq ft, it is of importance to evaluate the total light emission from the source. This unit is the lumen, a 1 c-p source delivering 1 lm to a square foot of area in setting up an illumination of 1 ft-c. The light intensity on a square foot varies inversely as the square of the distance, since this is the way the spherical area varies with radius, but the total light flow through any spherical surface as a whole is constant, irrespective of the distance. Hence a 1 cp source always emits $4\pi = 12.57$ lm. When light strikes a reflecting surface, the lighted surface varies in brightness with the angle of sighting provided the surface is "matte," but perfectly reflecting. Such a surface of small area illuminated by a source will not appear as bright as 1 cp even when of 1 sq ft area and illuminated by 1 lm. But such a matte surface lining the inside of a tower receives light not only directly, but by nearly perfect reflection from all other tower surfaces. Hence the light through a window may be measured in candle power by the number of lumens of direct illumination. A window of 1 sq ft facing a wall receiving 1 lm per sq ft will behave as a 1 cp source. The direct light from a fire will be of as many candle power as the rate of burning of oily wood is a multiple of the rate of burning of candle flame. Knowing the candle power and the total area, a candle-power per sq ft of front may be determined for the direct fire light.

Having set down the laws of light production and flow, it becomes of importance to discuss the sensitivity of human vision to faint light. Experiment suggests that on a clear, dark night at sea, a white light of 1 cp may be picked up with certainty at 3,200 yd. This is approximately the same thing as saying that such a light at 3,200 yd appears to be as bright as a 3.15 magnitude star from the astronomer's standpoint. The astronomer has set 6.0 magnitude as the brightness of a star which a person can just see when looking directly at it. It may be virtually impossible to pick up otherwise. There are cases where an observer could see a tower signal provided a tube had been set up permanently looking straight towards the tower. The tube limits the field of vision to one easily searched and stared at. It now becomes necessary to estimate the amount of fire-light and the brightness of all windows in terms of candle-power. That done, the distance of pick-up may be set.

The surface of a candle flame emits 3 cp per sq in. of surface. The hearth is such that at maximum a flame surface 2.5 ft sq could be obtained. This is about 650 sq in. At 3 cp per sq in., the fire has nearly 2,000 cp. This is high, so let it be assumed that the maximum does not exceed 1,000 cp, while 1 sq ft of fire seen through a window is about 430 cp. To get this amount of light, oily sticks must be completely and rapidly burned and the products of combustion must be removed. This must be done for several hours. The builder would install the best fireplace of which he might have the knowledge to insure such behavior. He would also make sure the fire would burn uniformly and well regardless of the direction of the wind. In other words, if he knew of what are called "modern" fireplaces he would use them; if he knew of chimneys, he would build one adequate for the purpose of a height greater than that of the peak of his roof. It was essential that he do so. In view of the primitive type of fireplace he did use, it must be considered proven that he knew of nothing better. This means that the tower must have been built before about 1400 A.D. The only possible escape from such a conclusion would be that this was a structure built primarily for eventual use as a church and that church rules and specifications had not advanced to the point of permitting such an innovation. It is well known that sacrificial knives were made of hard stone long after the stone age because no priestly group could bring itself to change the tabus of the past no matter what better material might be available. It seems improbable that hearths would come in this category, but not impossible.

Returning to the intensity of wall illumination, the fire of 1,000 cp emits about 6,300 lm towards the walls and floor and ceiling. The wall area is about 436 sq ft, the floor and ceiling about 539 sq ft. The total area is about 975 sq ft. The lumens per square ft of wall average (over wall, floor and ceiling) about 6.46. In view of the considerable area of floor and ceiling which may be non-reflecting, it is not advisable to assume a luminosity as high as 6.46 cp per sq ft of window facing wall area. A value of 4.5 might not be excessive and has been used. Window W4 with an area of about 0.84 sq ft will be a 3.8-cp source. Such a light can be picked up at 6,200 yd, or about 3 nautical miles. Points I and J can see the port. With some direction, it is possible that some of its light can be seen at K, but it is very unlikely it could be seen at L.

Now window W1 has an area of about 3.73 sq ft, so may rate as 16.8 cp. Its visibility is limited approximately to 13,100 yd or about 6.5 nautical miles. It is probable that K through Q can see this light, although the angle at Q is highly unfavorable, and so is it for L.

Window W2 has an area of about 6.43 sq ft, so may rate as 29.4 cp. It will be visible at 17,200 yd or about 8.5 nautical miles. This is more than ample for the assumed ship's course from A to hill B.

At the point of maximum firelight, the window area is about 1.6 sq ft. With the fire of intensity 432 cp per sq ft, the light amounts to about 690 cp. The actual vertical distance is not as great as the whole window height, so it would be more correct to set the light at, say, 400 cp. This light is good for about 32 nautical miles. The light could be seen with ease at Block Island, yet the ship would be about 4.75 miles or less than 10,000 yd from the tower. At point c, the firelight would appear as bright as a -1.0 magnitude star.

Window W3 is about 2.25 ft square, of 5.06 sq ft in area. Certainly a full 1,000 cp would be available in the direct firelight beam. This would yield a very bright light, relatively speaking. For wall light, this window is of about 22.8 cp, but at point c it would be only about a third of the window, or say 7 cp. The west window might be detected at point c (light visible to 4.2 miles instead of actual 4.75 miles) but could contribute little. There is an interesting question at this distance relative to the ability of an observer on a ship to separate the south window light from that of the west window. It can be shown that the distance is just about close enough to see two lights instead of one.

Two stars about 0.00032 radians apart can be just about separated. The angle here is about 0.00042 radians. So far as can be determined this may have been coincidence; it seems improbable that this might be intentional unless the builder did, as did the writer, make a model test. Point sources of light may be separated at 0.00032 radians, but finite sized lights such as these windows may require nearer to 0.00042 radians. The figuring is so close that it raises interesting questions. The 22.8 cp light is visible to about 15,300 yd, or 7.55 nautical miles. Points F, and perhaps C, could see the wall light. Points such as D, U, Z, G, AA, K, X and W can all see this light. Of course Tower Hill at E is in the line of direct firelight.

DISCUSSION

EDWARD ADAMS RICHARDSON[2].—The following points have been raised in private correspondence and will be discussed by the author.

1. The lowest placed window, W3, had its bottom edge so placed that the fireplace light might be seen at the harbor shore.

(A) In Fig. 2, the dot and dash line is drawn from direct fire, past the wooden sill to the shore of the harbor, proving that this consideration did govern the window location. This line is used to limit the height of the proposed outer wall parapet.

2. It is recommended that a trench be dug outside of the tower to locate any foundations of exterior construction. As a temporary apse might be added to the east, this direction is suggested.

(A) Mr. Godfrey dug a trench in 1948, from SSE to NNW starting nearly 100 ft outside of the tower and to the south. No traces of "proposed outer wall foundations" were found; hence, the author has assumed the design was made, but this wall was to be a later addition to the structure.

3. It is suggested that a mirror could be used for signalling from the windows.

(A) The author could imagine many methods of signalling, particularly to advise a vessel in the narrows of an error in its course, as by interrupting the west window light. The use of the fixed portions of the tower for signalling could be established, whereas auxiliary means could not, but could be assumed to be available. A mirror might have had some use, although this is doubted.

4. It is suggested that the windows of the first floor were splayed both outward and inward so light might be reflected from the illuminated inner splays to the outer, and whereby the window might be visible outside of the beam of direct fire-light.

(A) This suggestion is made, forgetting that after a vessel has passed through the whole range of direct fire-light, it can still see the window by the wall-light behind. However, near the extreme range rays, when the effective window width approaches zero, there might be some slight benefit from reflection from the splay surfaces. It is not seen that this was part of the design, however.

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[2] Publications Dept., Bethlehem Steel Co., Bethlehem, Pa.

5. It is suggested that the "hearsay" testimony of Gibbs to Lossing may have been fabricated by Gibbs. Certain statements of Gibbs were in error. He stated the columns rested on rock. Also, it seems odd that the story of Coddington seeing the tower in 1639 came down to Gibbs, not to the Coddingtons.

(A) If Gibbs did invent the story, it of course has no value. However, certain exceptions made to it are not warranted. Suppose someone did report that the tower was built on clay so rock-like in character that it could be excavated by pick only with difficulty. In the course of time, and lacking technical knowledge of the clay, the story will report a "rock foundation" and still not prove the total error of the tale. Other detail objections to the truth of the story seem to be in this category. In a period when sages are not learned by rote by each younger generation, it is fortunate if one member of a generation remembers the story. It need not be a blood-relation. As pointed out in the case of the Whittemore story, it went Whittemore, to Whittemore, to Newton, and to Richardson. Why should it be strange that a Gibbs rather than a Coddington should remember a Coddington story?

6. Erratum: Mr. Godfrey did not class the site as a whole as "undisturbed," merely certain parts of the "construction trench."

(A) This whole question of "disturbed" versus "undisturbed" is in an unsatisfactory state. The erratum more exactly represents the written views of Mr. Godfrey.

7. Erratum: The "construction trench" was not rock-filled, but it was circular and continuous under all columns, and was not only filled radially, but circumferentially only under each column.

(A) It is quite possible that the whole trench was started on the assumption that the tower must be erected on a relatively soft layer of clay and that the trench must be rock-filled throughout, at least on the bottom. However, after going down approximately 4 ft, it was found that a hard clay underlay the soft. The engineer might well have decided that in view of the hardness of the clay, the lack of workers and the difficulty in getting stone, that it would be sufficient to use piers to the hard clay. Due to the hardness of the clay, only a small amount of such material was excavated at each pier site.

8. Objection was raised that construction debris need not be found, provided the area was swept clean and a gravel floor was laid, a few traces of which were found.

(A) This is a reasonable objection.

9. It is suggested that Fig. 3 ignores wooden frames in the first floor windows. These were made of wood 4-in. square.

(A) It will be seen in Fig. 3 that all extreme rays may be determined by the splaying, as such rays do not go near wooden frames. Also, see the attention given to the wooden frame in Fig. 2. (See item 1). When it became evident later that the wooden frames might affect the range of direct firelight at W3 and W3 allowance was made for them.

10. It has been suggested that the tower was built as a "folly" by Governor Arnold, using the antique manner. It is further urged that in colonial times, a fireplace with wall ports instead of chimney was still in use when projecting eaves or other superimposed structure might make a chimney undesirable.

(A) For a man such as Governor Arnold, a "folly" might be merely a "front" for guiding smuggling vessels or privateers to port at night. This assumes, of course, that a man would choose such an expensive manner for creating a front. It also assumes that the tower was built by Arnold. But Arnold, if he ordered such a signalling system produced, knew about chimneys and should have insisted on one that would assure a fire however the wind might blow. The insistence on a colonial date for the building of the tower rests on such things as a "pistol" flint that might have been from a much earlier tinder box, a piece of pottery of negligible size, a footprint, a tiny piece of glass, and pieces of "clay" pipes. It is essential that these be proved lost by the builders, and also that it be shown that earlier persons could not have left equivalent material. It must be remembered that, if earlier peoples built the tower, it was left virtually undisturbed for nearly 300 yr; little or no change in levels would occur in that period, although much has happened in the shorter time since the colonial period.

11. Through a study of the perspective of tower pictures, it can be shown that the outer wall is not cylindrical, but slightly battered. In the same way, it can be shown that the tower has a slight tilt to the east.

(A) These items are of interest. It should be noted that an eastward tilt of the tower is consistent with partial observations by Mr. Godfrey. He noted that the plaster coating of the foundation stones under the east column had disintegrated. It was in this place that he found the stones loose enough to permit him to slip his arm into the pier up to the elbow. Now prevailing winds, particularly strong winter winds, are from the west or northwest; with a clay foundation, a somewhat greater pressure might be found on the clay to the east than that to the west. Over sufficient time, more water might be taken up by an overcompressed clay at the west than at the east, so, over a long period of time a noticeable tilt might be set up. During this action, loose stone might be slightly displaced at the east pier with consequent plaster disintegration. It is doubted that plaster cracking caused the tilt. It has been suggested that a windmill would add to the pressure and to the effect.

12. It is to be noted that in the paper, the structural analysis is based on a very simple method and assumes that the outer structure is so proportioned that the column loadings are central.

(A) Instead of limiting the floor beams to what they could carry as a uniform load (no wall support to flooring), suppose the flooring is simply supported at the wall and at the beams and, suppose further, that the load carrying ability of the beams is determined from the resulting distribution of loading span-wise. In this case, the gross floor loading raises to 123.1 psf or a net loading of 109.6 psf. This latter figure is greater than the original figure of 76.0 psf and is closer to the 100 psf live load for public buildings. a simpler assumption might be that each beam took a uniformly distributed load on a 6.15 by 17.48 section of flooring, the rest of the load being wall-supported. In this case, the net uniform loading rises to 98.7 psf. This is probably the simplification used by the tower designer.

These loads lead to increases in the total floor loads. Using 31000 lb per floor, it can be shown that the maximum stress in the columns becomes 74.32 psi, that is very close to the specification standard of 75 psi. This does not include the outer structure. Now, instead of aiming for a purely central loading of the column, suppose the exterior structure is increased to produce reverse bending moments. If the total load from the external roof is 66,000 lb, the extreme fiber stress falls from 74.32 psi to 71.39 psi, though the direct stress rises from 58.13 psi to 64.99 psi.

The above 66,000 lb from the outer structure assumes that the full bending moment of about 317,000 in.-lb is taken by the roof beams. This assumes sixteen roof beams 21 ft and 6 in. long or a structure diameter of about 66 ft inside of the outer wall. These proportions were used in Fig. 2.

teen roof beams 21 ft and 6 in. long or a structure diameter of about 66 ft inside of the outer wall. These proportions were used in Fig. 2.

It will be seen that the tower of Fig. 2 has been so designed that the columns will not be overstressed under partial loading even though that partial loading lasts indefinitely. Hence, we feel sure the tower was designed to be built in stages. It will be seen also that the design insures full use of the roof beam and roofing material, as this design stresses the beams to 317,000 in.-lb instead of the order of 258,000 in.-lb for the case originally given. The results of this disposition of material in the design is a structure with rather pleasing proportions.

In conclusion, it has been called to the attention of the author that H. B. Holand has written a paper[3] on a member of the Paul Knutson expedition who had the necessary character to give some feeling of face to the "builder of the tower." The "English scientist" is one Nicholas of Lynn, belonging to orders, who could use an astrolabe and magnetic compass, and who was rated as an astronomer. Although not so stated, he could have been trained in church construction and design. He studied magnetic declination from 54° N. on the Labrador coast and throughout the Hudson Bay region. He virtually located the north magnetic pole and he reported his discoveries in a paper, "De Inventione Fortunata," to the English king, Edward III. A globe by Frisius, of 1537, shows the Hudson Bay region, correct as to major characteristics; hence, someone must have made a thorough exploration, and the mapmakers benefited therefrom.

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[3] "An English Scientist in America 130 Years Before Columbus," by H. R. Holand, Transactions, Wis. Academy of Sciences, Arts, and Letters, Vol. XLVIII, Madison, Wis., 1959.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 3093

HIGHWAY LOCATION AND DESIGN BY PHOTOGRAMMETRIC-ELECTRONIC COMPUTER

By S. E. Ridge,[1] F. ASCE

SYNOPSIS

By combining photogrammetric techniques and electronic computers, the engineer is able to locate and design highways with economy and speed. Such equipment allows for a thorough study of the many alternatives available in highway design.

The integration of aerial photogrammetry and electronic computation into highway engineering provides the design engineer with powerful tools, which not only enable him to increase his productivity but also make it possible for him to improve the economy of his design. When used properly, these tools provide the design engineer with more information, more accurate information and provide that information in much less time than was ever before possible. Aerial photography, aerial photogrammetry, and electronic computation are of value in all phases of highway location, design, and construction; the area-study, the location-study, the final design, the preparation of the engineer's estimate and the final payment computation. They, at the same time, provide the highway engineer with material that is extremely valuable in giving the general public an understanding of his decisions in regard to route location.

Before going into detail, a brief mention should be made of the real purpose behind the effort to promote the adoption of these methods. Highway engineers are, of course, interested in saving engineering costs. Every dollar saved can

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[1] Ass't. Chf. Div. of Development, Office of Operations, Bur. of Pub. Rds., Washington, D. C.

be used for highway construction and the highway engineer's purpose is to build highways. Even more important, however, is the time saved. The states and the federal government have authorized the expenditure of highway funds at a rate far in excess of anything approached heretofore. This is all to the good, but these funds, as such, are of little value to the motorist. To be of value, they must be translated into in-place highway construction. Engineering is the first step in this translation process.

The construction cannot start until the engineering is complete. Past experience has shown that the engineering process, from fund authorization to contract letting, has in many cases taken as much as 2 yr. On one project only 27 weeks was required to move from authorization to contract. Six for preliminary line location, sixteen for design, and five for advertising and letting. This is far less than the 2 yr formerly required.

This is not, however, the most important reason for using these new tools and techniques. It is far more important to improve engineering. Engineers are professionals and as professionals, they should be more concerned about the quality of the work than the speed or cost of it. In addition, far greater savings in time and funds can be effected by performing a better location and design job than by performing that location and design job faster and cheaper.

A better location and design job means many things. It means the design of a project that can be constructed at a lower cost. A 1% or 2% reduction in construction costs is far more substantial than a similar reduction in engineering costs. It means a project that is a tenth of a mile shorter. A tenth of a mile reduction in the travel of the volume of traffic that will use a portion of the interstate system in the next 20 yr means an enormous saving of man hours and transportation costs. A better design can also mean a 1/2% reduction in the steepness of a grade for a mile or so. This could result in far greater savings of time and money than would the complete elimination of engineering.

Highway engineers are, therefore, using these new devices in order to improve their engineering analysis. By so doing, they can be of much greater service to their ultimate client; the highway user and the taxpayer.

The first step in highway location is the area-study. In the area-study phase, the entire area between the points of origin and destination is studied to ascertain the most acceptable corridor or corridors. High accuracies and excessive details are not necessary. In many cases, existing maps and photography will suffice for this study. If no photography is available or the available photography is too old to show the current development of the area, a semi-controlled mosaic made from photography of a scale of 1 in. to 2,000 ft should be acceptable. The photo-contour map is an excellent tool. Using this data, several alternate lines are studied in regard to probable traffic use, transport costs, construction costs, right-of-way costs, potential development of the area, etc., and the most suitable corridor or corridors are selected.

Where vertical control within the areas under study is not available, the airborn profile recorder (APR) is a most valuable instrument. The APR measures, by means of a radar beam, the distance between an aircraft and the ground, and translates this to actual elevation by means of a sensitive pressure altimeter. The horizontal position of the line is determined from synchronized air photos. With some control at each terminus of the flight line, this provides a very acceptable center-line profile for this stage of location. Such a profile, or profiles, also enables the photogrammetrist to ascertain the horizontal scale of the photography.

After the area-study has been completed, the next phase, the location-study, is begun. As the corridor of the proposed location has been reduced to a width of 1 mile or $1\frac{1}{2}$ miles, the photography for this phase can be taken at a scale of 1 in. equals 1,000 ft. This photography is usually a single strip exposed at 6,000 ft above the ground and covering a width of 9,000 ft. Another mosaic is prepared at a scale of 1 in. to 1,000 ft and a set of prints are made for stereo comparison. The necessary ground control for this photography (usually four points per photograph) is normally obtained by use of United States Geological Survey data extended into the flight strip by ground survey.

However, the first-order instruments now in use combined with the electronic computer can be used to bridge fairly long lengths of such photography. With good control at both ends of the flight strip, and some in the center, the first-order instruments can extend the control through the flight strip and check on the control existing at the other end. The computer can then distribute the error over the flight strip. Proper use of this method will produce ground-control throughout the flight strip acceptable for this phase of location.

The photography is used to study the soil existent along the lines, development of the area, drainage patterns and to locate aggregate sources, etc. The stereo model is used to study the ground configuration.

At the present time, there are two general methods of analyzing the topography. One is the preparation of a 1 in. equals 200 ft topographic map and the use of this map to provide ground elevation data on the several selected center lines. The center lines are laid out on the topographic map and the cross-sections are obtained by scaling the distance to the contours. These data are then analyzed in the electronic computer to produce the earthwork movement required for the selected grade lines. The use of the electronic computer allows the engineer to study far more lines and grades than was previously possible by hand methods.

Ordinarily, the electronic computer operation consists of several steps. The engineer selects the grades and vertical curve characteristics and the computer develops the center-line elevation at each cross-section station. These data together with the proposed cross-section of the highway are then fed into the computer to calculate the earthwork movement. Super-elevation and other refinements of design are not usually used at this stage.

A second method of analysis eliminates the need for a contour map at this stage by substituting for it a planimetric map and what is called a digital terrain model (DTM). The DTM is, in essence, a series of cross-sections eminating from a baseline roughly parallel to the center line at all points.

From an examination of the model, the engineer selects the center line he proposes to use and the horizontal curvature. This data is fed into the computer and the computer produces the stationing of each line of terrain data on the selected alinement, the skew angle between each line of terrain data and the center line, and a profile of the selected line. The computer can also produce profiles at any selected distance to the right or left, at the selected line.

The engineer then studies these profiles, which can be produced mechanically in the line plotter, and selects the stationing and elevation of the intersects of the vertical alinement and the vertical curve configuration he wishes to use. The electronic computer establishes the elevation of the trial grade line at each intersect with the terrain data.

These data together with the basic cross-section of the highway, data relative to the slope to use on cuts and fills of various heights, and compaction factors, are then fed into the computer and the earth movement in the form of a mass diagram and data regarding the limits of construction, and the elevation of the slope stakes on each side of the highway are produced. These data can also be mechanically plotted for visual inspection on the line plotter. The main difference in the calculation is that the cross-section areas are computed from skewed cross-sections rather than right angle cross-sections.

The engineer studies the results and makes his decision regarding the changes he desires in the alinement and/or grade, and the process is repeated until he has obtained an alinement that provides the best possible economy considering construction costs, right-of-way costs, transportation costs, and the general economic impact of the construction on the adjacent area.

Further development of this method is underway. It should be relatively easy to obtain data on right-of-way costs and to develop a program which would produce the approximate cost of obtaining the right-of-way for the selected line.

In addition, a program could be developed which, given the estimated traffic, would produce the cost of transportation over the selected line. With this data available to him, the engineer could produce, in a very short time, a very comprehensive analysis of the cost and value of the several lines investigated at the preliminary location phase.

The methods of obtaining these data for use in the DTM system are now fully developed. In 1956, the first device for the automatic translation of the analog information, produced by the photogrammetric instruments into tape or card records, was developed. Today at least five such instruments are in use. Generally, they consist of a manually operated keyboard on which the station of the cross-section or the X ordinate can be entered; a bar that can be set on the location of the X section in the model and which will then automatically register the offset from that line, the Y ordinate; and a device which will translate the vertical movement of the photogrammetric instrument into a true elevation, the Z ordinate. As the instrument is set on a point and adjusted, a button is pushed and the information is punched onto cards or a tape. A commercial model (TDT) is now available.

In addition, a device known as AUSCOR has recently been developed. This device automatically brings the images produced by the photographic pairs in the stereoplotter into focus. It has two principal uses. It can be used to automatically adjust the photographic pairs in the stereoplotter. This operation formerly required considerable time. The AUSCOR now performs this operation in a matter of minutes. After the photographic pairs are properly oriented in the photogrammetric instrument, AUSCOR automatically operates the vertical adjustment of the plotter and keeps the images in focus as the head is moved through the model. This enables the engineer to take terrain elevations with much greater speed than was heretofore possible.

With these two devices, the AUSCOR and the TDT, large numbers of terrain elevations can be obtained and recorded in a very short time. This makes it possible to economically record much more detailed terrain information on much larger areas. This increases both the accuracy and scope of the line investigation.

These data can be recorded and stored on cards or paper tape. When a location through a particular area is to be studied, the paper tape or cards can be transferred to magnetic tape or a similar memory device and used in connection with the terrain model system. The use of the magnetic tape is suggested since, with it, the analysis of the selected lines can be accomplished with much greater speed than would be possible if the paper tape or cards were used directly.

CONCLUSIONS

We now have the equipment and the methods necessary for a much more detailed and much faster analysis of possible highway locations at the preliminary location stage. With this equipment and with these methods, it should be possible for the highway engineer to improve his engineering analysis and to thereby produce far better and more economical highways. It is up to the highway engineers to put these new techniques and tools into use and to obtain the benefits possible from their use.

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COLUMBIA BASIN STREAMFLOW ROUTING BY COMPUTER

By David M. Rockwood,[1] M. ASCE

With Discussion by Messrs. Willard M. Snyder; and David M. Rockwood

SYNOPSIS

A method for using an electronic digital computer for streamflow routing in the Columbia Basin is described. Water excesses of rainfall or snowmelt can be routed by sub-basins to synthesize streamflow, which in turn is routed through lakes and channels to downstream control points. A new routing technique is made possible by the use of the computer.

INTRODUCTION

Among the many types of engineering problems involving great masses of computations are those encountered in hydrologic practice for synthesizing streamflow. This paper describes a method developed specifically for deriving streamflow variation in the Columbia River Basin. The method was designed primarily for providing a direct and objective streamflow forecasting procedure for periods up to 10 days in advance, on the basis of preceding streamflows and forecasts of basin inputs from snowmelt or rainfall. It may also be used for design flood determinations or reservoir regulation studies. The procedure utilizes a new technique for streamflow routing that is made practical by the capabilities of a medium speed electronic digital computer.

The primary capabilities of electronic digital computers, when applied to streamflow forecasting, are their ability to, (1) handle large amounts of input data, which are used to define the initial streamflow condition and forecasts

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of hydrometeorological events, (2) perform rapid arithmetic computations, for routing streamflow in small finite increments of time and storage, (3) follow a predetermined series of instructions, which automatically directs the arithmetic and data processing operation, (4) store numerous digital values representing basin, channel, or lake routing coefficients for use as directed by the basic program of instructions, and (5) provide digital output values successively throughout the operation, which are used for defining forecasts of streamflow at key gaging stations.

PROGRAM DEVELOPMENT

The program was developed in the office of Division Engineer, U. S. Army Engineer Division, North Pacific, which supervises the planning and operation of several major water control projects in the Columbia River Basin. Daily reservoir regulation problems as well as project design computations require the use of a streamflow routing technique for the entire basin. The problem involves synthesizing streamflow for the component sub-basins lying within the Columbia River Basin, and routing streamflow through lakes, reservoirs, and channels in order to establish predictions of flow at a number of key downstream gaging stations.

The essential requirements for this procedure are the following: (1) the time increment must be small enough to represent fluctuations of streamflow which occur in the Columbia River Basin tributaries (normally, 6 hr); (2) the total time for preparation of forecasts, from receipt of basic meteorologic and hydrologic data to derivation of forecasted flows should be less than four hrs; (3) the method should provide for adjusting streamflow values periodically (usually daily) in accordance with observed conditions; and (4) the method of applying forecast values of input of water supply (either in the form of rainfall or snowmelt excesses, or streamflow) should be flexible, and allow for applying more than one forecast condition for any forecast period.

During August 1956, the North Pacific Division Office obtained use of a type 650 IBM Electronic Digital Computer, with certain peripheral equipment, primarily for computation of system power studies for the Columbia River Basin. This computer is adaptable to the solution of streamflow routing problems in accordance with the requirements set forth above. The program enables the computer to solve the storage routing equations in many successive small increments, combine computed tributary and local inflows successively downstream, and provide values of routed flows at all specified gage locations. The operation is entirely automatic and is controlled by proper sequencing of input values. In effect, when all storage constants have been evaluated, the program provides a model of the Columbia River System that can be used to represent streamflow for any specified meteorological or storage condition.

The program has been "checked out," and sufficient storage routing constants have been derived to test it on a day-to-day forecasting basis during the 1957 spring snowmelt flood. Streamflow forecasts were made on 35 consecutive days. These forecasts were used to help establish regulation of streamflow at reservoirs (primarily Grand Coulee Dam) for flood control operation on the main stem of the Columbia River. This trial showed the method to perform as desired, and results to be available within prescribed time limitations.

COLUMBIA RIVER BASIN STORAGE

There are three types of storage delay to runoff that must be evaluated in the Columbia River System. These are, (1) basin storage, whereby the streamflow at an upstream gaging station can be computed as a time-rate function from basin input values of rainfall or snowmelt excesses on the tributary areas, (2) lake storage, resulting from the many large lakes in major tributaries, whose effects on streamflow may be evaluated by the basic reservoir-type storage equation, and (3) channel storage, which results from the natural delay of flow through channel reaches. There are several commonly used methods for evaluating these storage effects. Except for lakes or reservoirs for which storage is a direct function of outflow, the methods are empirical in nature, in that they depend on trial-and-error procedures for establishing or verifying basin or channel time-delay characteristics. Unit hydrographs and various channel routing techniques fall within this category.

ROUTING METHOD

The program developed as described[2] herein relies on a routing method which is basically in the form of the general storage equation

$$I_t = O_t + \frac{dS}{dt} \quad \text{.................. (1)}$$

in which I_t and O_t are inflow and outflow, respectively, in cubic feet per second, and dD/dt is the rate of change of storage at time, t. For cases where storage is a function of outflow (as in natural lakes, or for channel storage for short reaches where wedge storage is negligible in comparison with prismatic storage)

$$S = T_s\, O \quad \text{.................. (2)}$$

in which T_s is the proportionality factor between storage and outflow. Differentiating Eq. 2 with respect to time,

$$dS/dt = T_s\,(dO/dt) \quad \text{................ (3)}$$

Substituting this expression in Eq. 1, it becomes

$$I_t = O_t + T_s\,(dO/dt) \quad \text{..............(4a)}$$

or

$$dO/dt = \frac{I_t - O_t}{T_s} \quad \text{................(4b)}$$

which represents the form of the storage equation used in this method. For natural lakes, the value of T_s is not constant, but it can be evaluated from the storage and outflow characteristics. This can be done by evaluating the differential of Eq. 2 with respect to h,

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[2] "An Electronic Analog for Multiple Stage-Reservoir-Type Storage Routing," by David M. Rockwood, and Carver E. Hildebrand, Project CW-171, Tech. Bulletin 18, Corps of Engrs., Civ. Works Investigations, March, 1956.

$$T_s = \frac{dS/dh}{dO/dh} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

in which T_s for a given elevation, h, is given in units of time, dS/dh represents the slope of the storage-elevation curve, and dO/dh is the slope of the

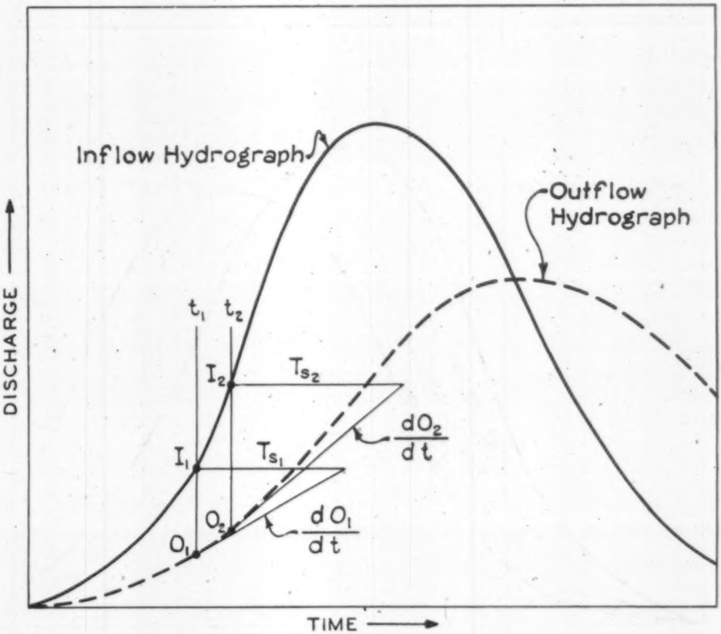

FIG. 1.—LAKE STORAGE EVALUATION

discharge-elevation curve at elevation h. From Eqs. 4 it can be shown that with zero inflow, the outflow recession is in the form

$$O_t = O_o e^{-t/T_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

in which O_o is the initial outflow at time, t = 0, O_t is the outflow at time, t, and T_s is the proportionality constant previously defined, corresponding to the value of O_t. Eq. 6 is the typical decay-type function characteristic of streamflow recession, but with a varying recession coefficient for this method.

In application to natural lakes or reservoirs, Eq. 4 may be used in the program in small finite increments of time, as shown schematically in Fig.

1. The same general equation may be applied to channel storage by reducing the length of the channel reach to the point where "wedge" storage is negligibly small in comparison with "prismatic" storage. By successive routings (herein called "phases") through many small increments of reservoir-type storage, the time delay from channel storage can be evaluated as shown in Fig. 2. Similarly, basin storage can be evaluated through successive "phases" of reservoir-type storage, to empirically represent the effect of basin storage.

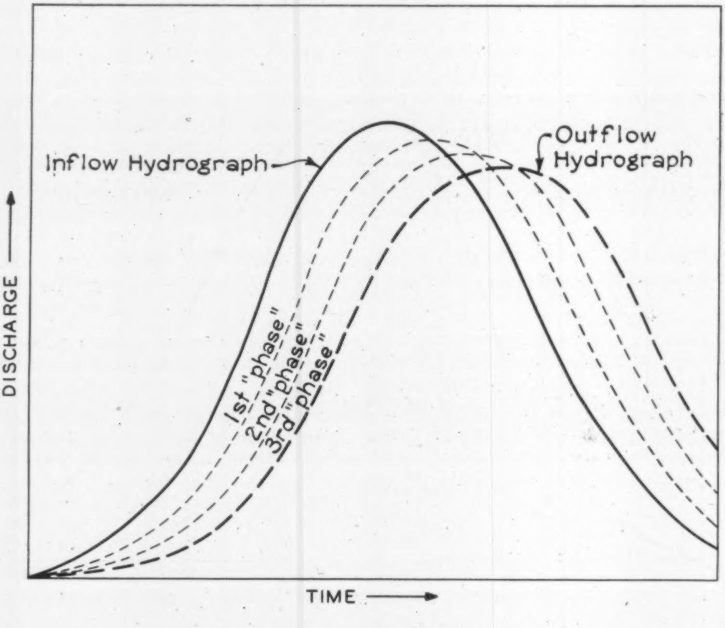

FIG. 2.—CHANNEL STORAGE EVALUATION

This distributes runoff from basin input values in inches per 6-hr period, which provides streamflow in terms of cubic feet per second, comparable to that obtained through use of unit hydrographs. The storage routing method used herein has the advantage of being able to adjust values to observe conditions on a day-to-day basis. Fig. 3 shows, schematically, the distribution of runoff by successive routings of a unit basin input through two "phases" of reservoir-type storage.

An important feature of the routing method for basin and channel storage is that the time of storage may be varied with flow. This, in effect, results in distribution of runoff from snowmelt or rainfall excesses that may be varied with the flow condition. For example, the time delay to runoff may be made shorter with high rates of runoff and longer with low rates of runoff. Similarly, the time of travel of flood waves through channels may be made to vary with discharge, according to channel conditions. This feature allows great flexibility in empirical fit of data from constants developed from actual record, and the method may be used to closely represent the actual time delay to runoff.

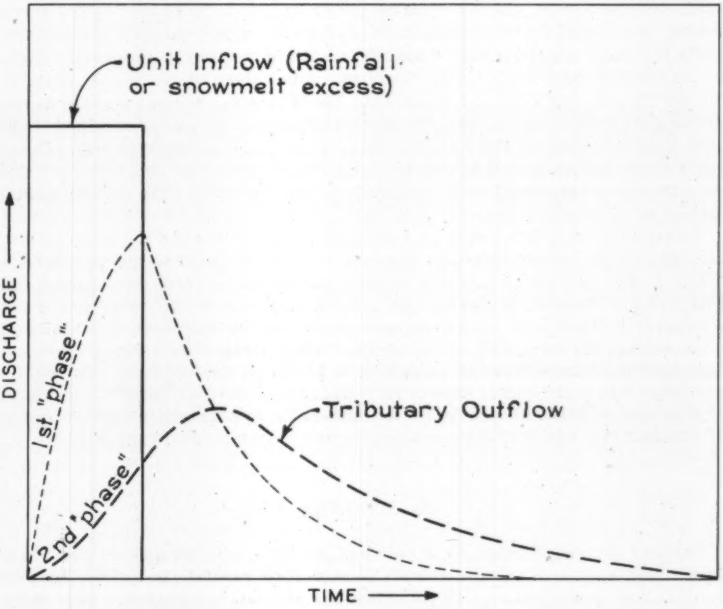

FIG. 3.—BASIN STORAGE EVALUATION

BASIC CODES AND ROUTINES

The basic code comprises a set of instructions that directs the computation of streamflow from input data read from punch cards, through appropriate routing sub-routines and summing routines. The three routing sub-routines, for evaluating basin, channel, and lake storage, are based on evaluation of the storage equations as previously explained. The sub-period time increment for each phase of routing is 3 hr. Constants that are proportional to each of the storage times for each successive routing are used and stored for each basin area or channel reach. For evaluation of basin storage, inflow amounts are divided between surface (direct) and subsurface (groundwater) runoff, and each component is routed separately through two phases of reservoir storage.

All necessary constants for basin, channel, and lake storage for the entire basin, together with the necessary programs and routines for the actual routing, are contained on a single loading of the storage drum of the computer, (2,000 ten-digit word storage capacity). Sufficient space is reserved for obtaining outflow data for a 10-day forecast, with values of discharge given for four times per day (at 6-hr intervals). There are sixty-eight stream gaging locations or local inflow points, for which discharge data can be obtained. Input cards may be read in for basins, with values given as snowmelt or rainfall excesses in inches per 6-hr period, and output cards are punched out which give values of streamflow in cubic feet per second for the stream gaging point.

This operation is successively repeated downstream, and tributary inflows are added and routed through channel or lake storage to the downstream points. A set of "pilot" routines directs the operation from one reach to the next, and ties the entire routing into a completely automatic operation. Thus, successive inflows are routed and summed from the headwaters of the basin to the outflow for Columbia River at The Dalles, Oreg. For a 5-day forecast, flows for the entire basin can be routed in 3/4 of an hour of IBM 650 machine time. Fig. 4 is a schematic master flow diagram of the Columbia River Basin, showing the relative locations of the sub-basin areas, major lake storages, and channel reaches that are presently incorporated in the program.

INPUT DATA

For any drainage basin or local ungaged area, inflows may be read in as basin runoff excesses resulting from snowmelt or rainfall, in inches per 6-hr period, or as basin outflows in cubic feet per second. In addition it is possible to enter any tributary at one of the downstream gaging points, with given or assumed values of streamflow in cubic feet per second, without necessitating routing through the tributaries upstream from the control point. Thus, it is possible to route flow in any section of the river from given natural or regulated flows at the upstream control points.

Snowmelt is computed from basin snowmelt indexes. For the 1957 trial operation, a simple maximum temperature index of snowmelt was used, with provision for an effectiveness factor to account for variations in albedo of the snow surface. Snow investigations[3] have been summarized, which describe the theory and application of snowmelt indexes. Estimated or observed values of snow cover area, in percent of total drainage area at the gage, are required.

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[3] "Summary Report of the Snow Investigations, Snow Hydrology," Corps of Engrs., North Pacific Div., U. S. Army Engr. Div., North Pacific, Portland, Oreg., Chapters 5, 6, and 9, June 30, 1956.

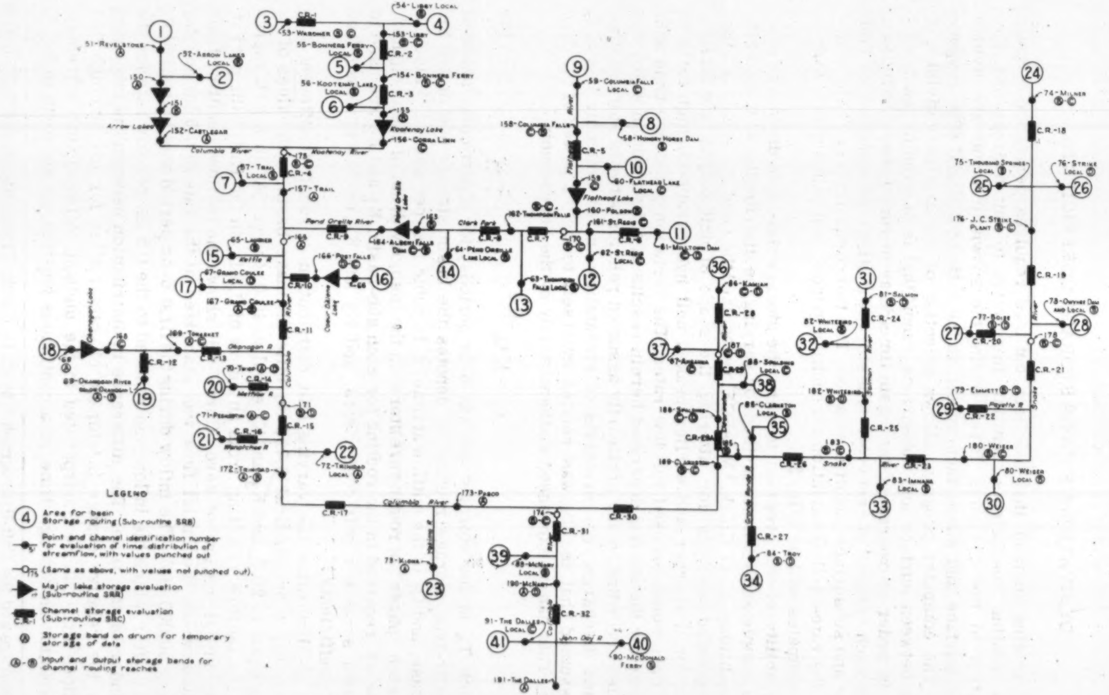
51-Revelstoke
52-Arrow Lakes Local
150
151
Arrow Lakes
152-Castlegar
Columbia River
Kootenay River
C.R.-1
53-Wardner
54-Libby Local
153-Libby
55-Bonners Ferry Local
C.R.-2
154-Bonners Ferry
56-Kootenay Lake Local
C.R.-3
155
Kootenay Lake
156-Corra Linn
175
57-Trail Local
C.R.-4
157-Trail
Pend Oreille River
Pend Oreille Lake
163
164-Albeni Falls Dam
C.R.-9
165
65-Laurier
Kettle R
166-Post Falls
Coeur D'Alene Lake
C.R.-10
66
67-Grand Coulee Local
167-Grand Coulee
C.R.-11
168
Okanogan Lake
68
69-Okanogan River
C.R.-12
169-Tonasket
C.R.-13
Okanogan R
70-Twisp
C.R.-14
Methow R
171
71-Peshastin
C.R.-15
C.R.-16
Wenatchee R
72-Trinidad Local
172-Trinidad
173-Pasco
C.R.-17
Columbia River
73-Kiona
59-Columbia Falls Local
158-Columbia Falls
58-Hungry Horse Dam
C.R.-5
60-Flathead Lake Local
159
Flathead Lake
160-Polson
162-Thompson Falls
Clark Fork
C.R.-8
C.R.-7
64-Pend Oreille Lake Local
170
161-St Regis
C.R.-6
61-Milltown Dam
62-St Regis Local
63-Thompson Falls Local
C.R.-30
174
C.R.-31
89-McNary Local
190-McNary
C.R.-32
91-The Dalles Local
John Day R
90-McDonald Ferry
191-The Dalles
86-Kamiah
C.R.-28
187
87-Anatone
88-Spalding Local
188-Spalding
C.R.-29
C.R.-29A
186
189-Clarkston
85-Clarkston Local
Grande Ronde R
C.R.-27
84-Troy
C.R.-26
Snake
81-Salmon
C.R.-24
82-Whitebird Local
182-Whitebird
Salmon River
C.R.-25
183
River
C.R.-23
83-Imnaha Local
180-Weiser
80-Weiser Local
79-Emmett
C.R.-22
Payette R
C.R.-21
Snake
77-Boise
C.R.-20
178
78-Owyhee Dam and Local
River
C.R.-19
176-J. C. Strike Plant
75-Thousand Springs Local
76-Strike Local
C.R.-18
74-Milner
LEGEND
Area for basin Storage routing (Sub-routine SRB)
Point and channel identification number for evaluation of time distribution of streamflow, with values punched out
(Same as above, with values not punched out).
Major lake storage evaluation (Sub-routine SRR)
Channel storage evaluation (Sub-routine SRC)
C.R.-1
Storage band on drum for temporary storage of data
Input and output storage bands for channel routing reaches

FIG. 4.—SCHEMATIC MASTER FLOW DIAGRAM

Rainfall amounts, which may contribute to runoff during snowmelt floods, are estimated from reporting network observations.

DERIVATION OF BASIN STORAGE COEFFICIENTS

The routing method that has been outlined requires derivation of basin storage routing coefficients, similar in function to unit hydrographs. For application to the Columbia River Basin, the snowmelt inputs are divided between surface and subsurface runoff, in the ratio of 20% and 80%, respectively. The computer program allows selection of any arbitrary division of inflows between surface and subsurface runoff that is desired for each sub-basin. In order to represent the basin time delay to runoff, two coefficients (one for each "phase" of reservoir-type storage routing) are required for both surface and subsurface runoff evaluation. The two coefficients are able to describe the rate-of-flow variation for each component, just as many unit hydrograph ordinates will do (Fig. 3).

The ability of the program to vary the storage time with discharge provides a simple and realistic method for varying the distribution of runoff with flow conditions. C. O. Clark,[4] M. ASCE, recognized the need for such variation and suggested methods for altering the shape of unit graphs, and thereby provide for a sharper and earlier peaked unit hydrograph for high rates of runoff than would be used for low rates. The variation of storage time with discharge for the method described herein results in a distribution which can be varied according to any arbitrarily assumed relation between storage time (T_s) and discharge. For synthesis of streamflow in the Columbia Basin, it was assumed that the T_s was related inversely to the 0.2 power of the discharge. This can be expressed mathematically by the relationship

$$T_s = K\,Q_t^{-0.2} \qquad (7)$$

in which T_s is the storage time (in 6-hr periods) for a particular phase of reservoir-type storage routing, K denotes the basin storage coefficient for that phase, and Q_t is the outflow at time, t, in cubic feet per second. Values of K for each "phase" of routing are stored in the memory drum of the computer, for use as required in the routing for each sub-basin. K_1 and K_2 are used to represent surface runoff coefficients, and K_3 and K_4 represent subsurface runoff coefficients.

Fig. 5 illustrates the varying unit distribution of runoff at different discharges for a hypothetical 3,790 sq mile drainage basin, with values of K_1 and K_2 equal to 20.0, and K_3 and K_4 equal to 40.0. Curve "A" in Fig. 6 represents the outflow resulting from an inflow of 0.5 in. in the first 6-hr period, with an initial outflow of zero. Curve "B" shows the outflow resulting from the case where the initial flow was stabilized at the rate of 0.5 in. per day (equal to 50,000 cfs). The inflow during the first 6-hr period was increased by 0.5 in., after which the inflow was reduced to the 0.5 in. per day rate for the remainder of the time. The difference in distribution between the two cases shows the relative change in storage characteristics for low and high flow conditions that can be incorporated in the method. Between such extremes, the variation in storage time is a continuous function, as set forth in Eq. 7.

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[4] "Storage and the Unit Hydrograph," by C. O. Clark, Transactions, ASCE, Vol. 110, 1945, pp. 1419-1446.

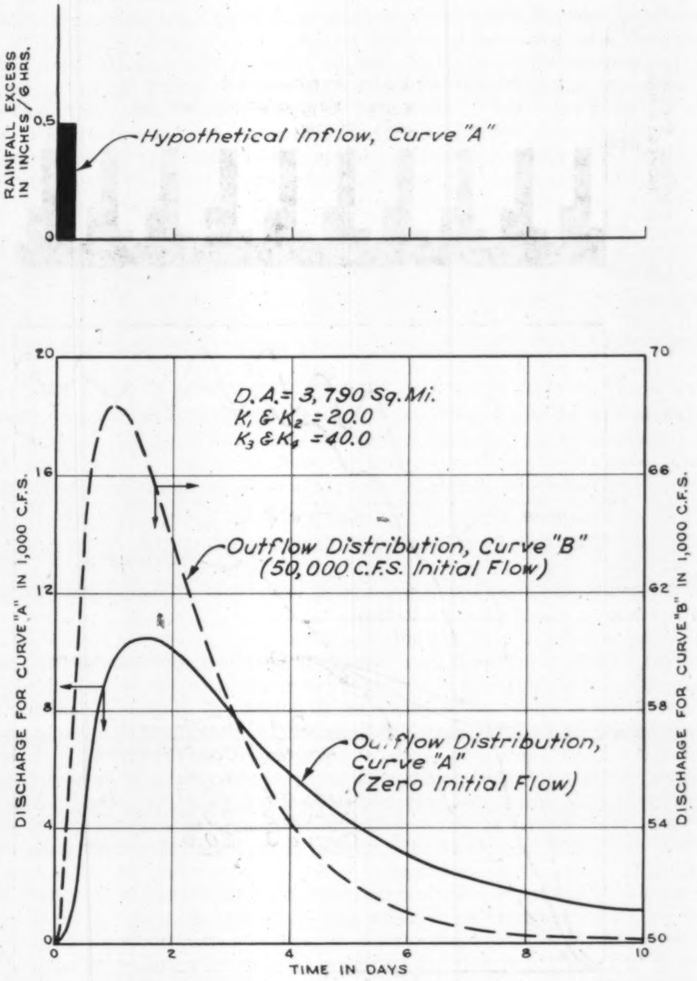

FIG. 5.—COMPUTED BASIN OUTFLOW FOR HYPOTHETICAL DRAINAGE, SHOWING VARIATION IN UNIT DISTRIBUTION FOR HIGH AND LOW FLOW CONDITIONS

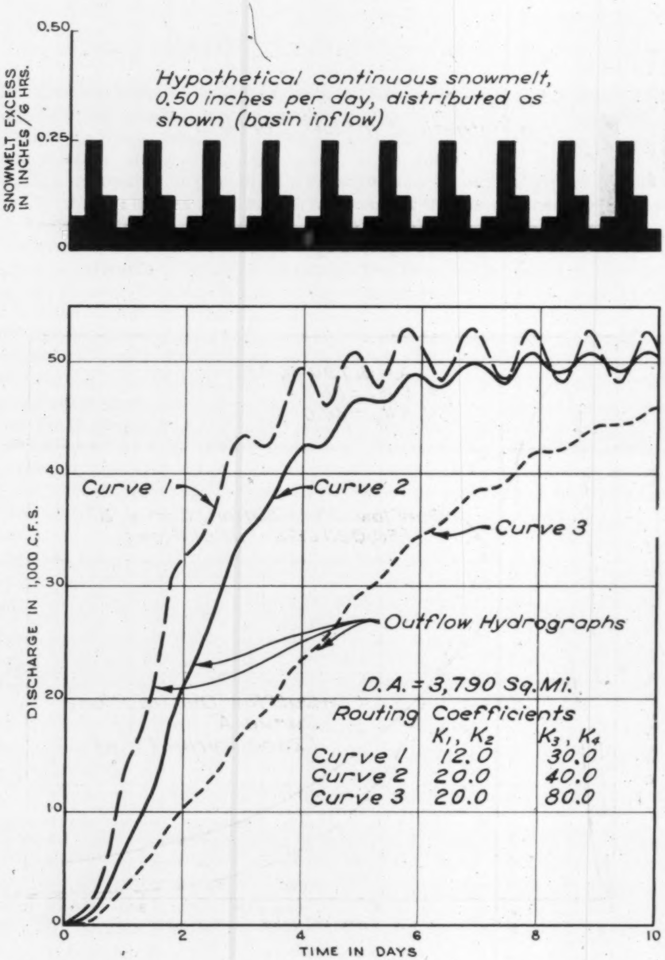

FIG. 6.—COMPUTED BASIN OUTFLOW HYDROGRAPHS FROM CONTINUOUS SNOWMELT INPUT FOR HYPOTHETICAL DRAINAGE

Fig. 6 shows the computed outflow hydrographs that would result from 10 days of continuous snowmelt, with an initial flow of zero. The inflow was made to vary in a daily fixed percentage pattern of snowmelt, in order to account for its diurnal variation. The resulting outflow hydrographs were computed using three different sets of basin routing coefficients, as shown in Fig. 6. These curves illustrate how the basin storage coefficients may be varied to represent the runoff characteristics of a given basin. They also show that the diurnal fluctuation in streamflow resulting from daily snowmelt variation can be reproduced by this method. With greater storage delay, this effect tends to be "dampened" out. The outflow hydrographs in Fig. 6 are similar to S-curves developed from unit hydrograph computations. The diurnal fluctuations should not be confused with the fluctuations that sometimes occur from S-curve computations, resulting from incorrect proportionment of unit hydrographs.

Actual values of basin routing coefficients are determined by trial and error methods in reconstituting observed conditions of streamflow. Reconstitutions thus far have been limited to sub-basins of the Columbia River, for which snowmelt is the principal source of water supply. The computer program also makes provision for adding base flow amounts to the derived runoff from surface and subsurface flow. The routing technique has been tested sufficiently to show its feasibility and acceptable accuracy of results. For streamflow forecasting, the method has the distinct advantage of being capable of easy adjustment to observed flow conditions on a day-to-day basis. This would be extremely cumbersome using unit hydrograph procedures.

DERIVATION OF CHANNEL STORAGE COEFFICIENTS

Having developed the concept of using many small segments of reservoir-type storage to represent the storage effect of natural stream channels, it is then necessary to develop methods for applying it to specific channel reaches. It is convenient to consider each channel reach as representing a certain amount of storage time (usually in hours) for which a flood wave is transposed by traveling through it. In addition to the transposition, there is also modification of form of the flood wave, by which the wave (or any time irregularity of inflow) is "flattened" or modified in shape with respect to time. In the commonly used "Muskingum" flood routing equation,[5] the value of "K" determines the amount of transposition of a flood wave, while the value of "x" (the ratio of the storage as a function of inflow to that as a function of outflow) determines the modification of wave form. The Muskingum "K" value is expressed in hours, and where "x" = 0, it is equivalent to the value of T_s. The modification of wave form in the Muskingum method is dependent on a derived value of "x" for each channel reach. The limits of "x" range from 0.5 for the case where no modification occurs, to 0.0 for the maximum modification of wave form that occurs when storage is a direct function of outflow. The values of "x" for natural stream channels are normally between 0.20 and 0.35.

In multiple-phase reservoir storage channel routing, the same general technique applies, in that the total transposition for a given reach is represented by the storage time, T_s. The number of "phases" of reservoir storage in a particular reach determines the degree of modification of the wave form. With only one "phase," the modification is a maximum and corresponds to use of "x" = 0 in the Muskingum equation. With a large number of "phases," equivalent to an assumed routing time of 1 hr per phase, pure translation of the flood wave results and is equivalent to use of "x" = 0.5 in the Muskingum method. Intermediate between these extremes, any number of "phases" may be used to represent the wave modification for a given channel reach.

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[5] "Routing of Floods Through River Channels," Corps of Engrs., Chf. of Engrs., Engineering Manual, Civil Works Construction, Part CXIV, Chapter 8, September, 1953.

Fig. 7 represents the variation in wave modification that may be obtained through use of this technique. An assumed arbitrary inflow was routed through a total of 36 hr of time, by use of 1, 3, and 6 "phases" of reservoir storage, respectively. It is seen that the greatest modification occurs with a single "phase" of 36 hr. Curve "A" shows the effect of routing through an uncontrolled lake or reservoir with that particular storage time. Curves "B" and "C" show the computed outflows with 3 and 6 "phases," respectively. In this case, the use of 3 "phases" produced results that were equivalent to use of "x" equal to about 0.35 in the Muskingum flood routing equation.

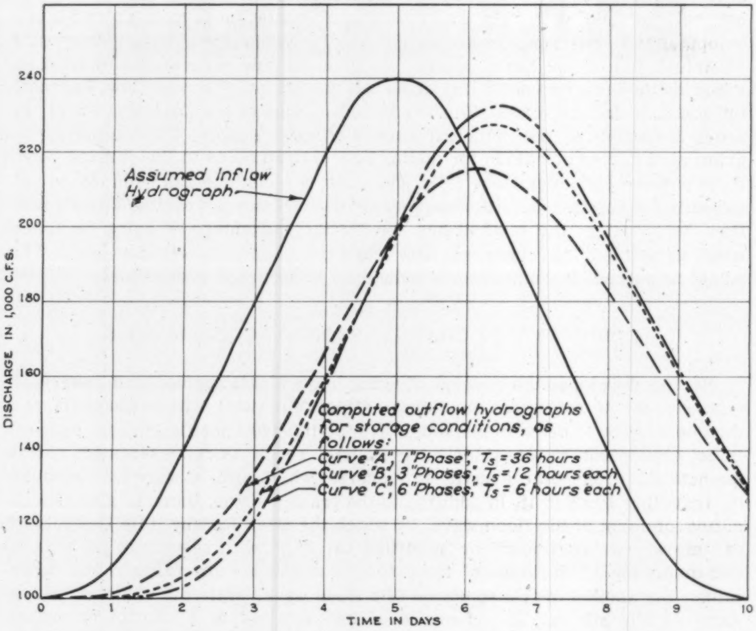

FIG. 7.—CHANNEL STORAGE EVALUATION FOR HYPOTHETICAL INFLOW, SHOWING EVALUATION OF OUTFLOW THROUGH VARYING NUMBERS OF ROUTING PHASES

For the Columbia River Basin, channel reaches were selected from gaging stations and key control points, and they are generally from 20 miles to 50 miles long. Three "phases" of reservoir storage are used for each reach. The mean storage time for each "phase" of channel routing on the main stem is about 3 hr. The routing coefficients were derived from general knowledge of travel times in the channel reaches, and adjustments were made by trial and error procedures through reconstituting past record.

The sub-routine for channel routing also incorporates the ability to change the storage time as a function of discharge. For the Columbia River, the storage time was allowed to vary inversely with the 0.2 power of the discharge. Thus, doubling the discharge results in about a 13% reduction of storage time. The program is flexible and would allow any arbitrary function of storage time with discharge. The computer operation of "Table Look Up" is used for this purpose. This technique also has the advantage of doing away with the unrealistic "dips" in outflow resulting from a sudden rise in inflow, which is characteristic of the Muskingum method.

DERIVATION OF LAKE ROUTING COEFFICIENTS

Lake routing coefficients were determined directly, by evaluating T_s through use of the relationship given in Eq. 5 for a number of elevations. This establishes the relationship between storage time and discharge. For the computer program twenty-five such values were determined for each lake to cover the range of flows normally experienced. "Table Look Up" operation was used to select the value of storage time corresponding to a given outflow in the routing sub-routine. Table 1 illustrates the variation of storage time (T_s) with discharge for Kootenay Lake, which is one of the major lakes in the Columbia Basin, in units of either days or hours. It is noted that the storage time decreases markedly with increasing discharge. It will also be noted that the storage time of this lake is in the order of 10 days. The channel storage time in the main stem of the Columbia River is in the order of 6 miles per hr,

TABLE 1.—VARIATION OF STORAGE TIME WITH DISCHARGE FOR KOOTENAY LAKE

Discharge, in cubic feet per second	Storage Time	
	Days	Hours
10,000	20.6	495
20,000	16.8	402
30,000	13.9	334
40,000	11.2	268
50,000	10.2	244
70,000	9.16	220
100,000	8.53	205
150,000	6.24	189

or 140 miles per day. Obviously, the major lake storage in the Columbia Basin constitutes a much greater time delay to run off than channel storage.

SUPPLEMENTARY ROUTINES

In addition to the main routing routine with its subroutines and pilot routines, two supplementary routines were written that are entirely independent of other routines. These are for the purpose of reducing the time necessary for preparation of a forecast. The first is the "distribution routine" and is used to distribute daily values of basin snowmelt or rainfall runoff excesses into 6-hr increments, according to a percentage pattern which may be changed with time. The second supplementary routine is used for plotting the outflows in cubic feet per second for the end of each 6-hr period, in the form of hydrographs, on the computer's tabulating machine.

CONCLUSIONS

The purpose of this description is to present the over-all scope and basic routing method for the procedure. The many details of programming and coding, as well as the data processing techniques required to provide a workable solution in the time normally available for streamflow forecasting, have not been mentioned. The method provides the framework for completely automatic determination of the components of streamflow in the Columbia River Basin, whereby all hydrometeorological factors affecting runoff may be evaluated objectively through cause and effect relationships. The method is feasible only with an electronic computer, but with the general advent of computers, it provides the hydrologist with a flexible and rapid technique for synthesizing streamflow.

ACKNOWLEDGMENTS

This procedure was developed under the general supervision of Mark L. Nelson, F. ASCE, Head, Water Control Branch, U. S. Army Engineer Division, North Pacific. The writer wishes to acknowledge the work done by Edward M. Davis of that office in coding the basic program and supplementary routines, and for general aid in data processing techniques. His work contributed immeasurably to the successful completion of the procedure.

DISCUSSION

WILLARD M. SNYDER,[6] M. ASCE.—In the description of the routing equations used in the Columbia Basin the author states that routing coefficients used in the computer program were determined by trial and error. An outline of more direct procedure resulting from modifications to an available method[7] may be useful to engineers engaged in routing studies.

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[6] Staff Research Hydrol., Tributary Watersheds Program, TVA, Knoxville, Tenn. Formerly, Head, Hydrology Sec., Hydr. Data Branch, Knoxville, Tenn.

When "inflow" and "outflow" data are available, the pattern of the relationship, that is, the time-distribution coefficients can be determined by the method of least squares. There is little difference, in applying this technique, between distribution coefficients that represent a unit hydrograph, or distribution coefficients that serve as routing coefficients through a reach. A big advantage in using the method of least squares is that computer programs for this method are readily available for machines where subroutine libraries have been accumulated.

The method of derivation of distribution coefficients can probably best be illustrated by a numerical example. Fig. 8 shows the hydrographs for Chestuee Creek at Dentville, Tenn., and, upstream above a junction point, for Chestuee Creek at Zion Hill and Middle Creek below Highway 39. These hydrographs show the flood of November 1957.

Chestuee Creek, a tributary of the Hiwassee River, drains a watershed of 114 sq miles above Dentville. Drainage areas above the Zion Hill and Middle Creek stations are 37.8 and 32.7 sq miles, respectively. These gaging stations are operated by the Tennessee Valley Authority (TVA) under its Tributary Watershed Program.

The time interval used in plotting Fig. 8 was 4 hr. This time interval was maintained throughout the analysis. Runoff volumes for the area between the Dentville gage and the two upper gages were estimated, using a rainfall recording gage located in this intermediate area. These runoff volumes are plotted at the top of Fig. 8.

The obvious analytical problem presented in Fig. 8 is to predict the Dentville outflow hydrograph from the mixed inflow consisting of runoff from the intermediate area and inflow into the heads of the reach. A prediction equation is necessary, designed so that, when fitted to the data, it will produce the desired distribution coefficients.

The first step in preparation of the analysis was to add the Zion Hill and Middle Creek hydrographs to produce a single hydrograph of reach inflow. This was done for simplicity and because of approximately equal time of travel. It is not necessary to the solution. The second step in the analysis was to devise two numeric functions which are first approximations of the distribution coefficients for the two types of inflow. The choice is not critical and any reasonable approximation from hydrograph inspection will suffice. In Table 2 are listed the two numeric functions that were assumed. The numeric functions were used to generate so-called "independent" variables for the prediction equation in a manner that will subsequently be described. The outflow hydrograph is, of course, the "dependent" variable.

The successive ordinates of the outflow hydrograph were assumed to be given by computation on successive runoff and inflow quantities as illustrated by Eqs. 7

$$O_4 = w_1(5\ RO_4) + k_1(1\ I_4) \qquad \text{(7a)}$$

$$O_8 = w_1(5\ RO_8 + 10\ RO_4) + w_2(5\ RO_4) + k_1(1\ I_8 + 2\ I_4) + k_2(1\ I_4) \qquad \text{(7b)}$$

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[7] "Hydrograph Analysis by the Method of Least Squares," by Willard M. Snyder, Proceedings, ASCE, Vol. 81, Paper No. 793, September, 1955.

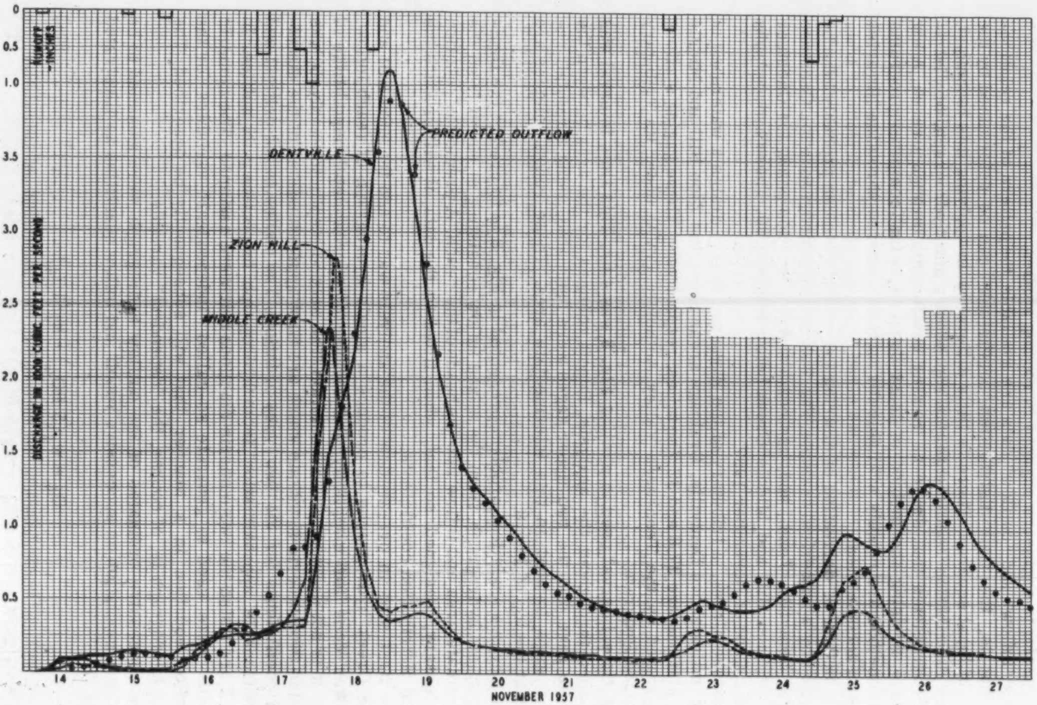

FIG. 8.—INFLOW AND OUTFLOW HYDROGRAPHS FOR CHESTUEE CREEK

$$O_{12} = w_1(5\ RO_{12} + 10\ RO_8 + 8\ RO_4) + w_2(5\ RO_8 + 10\ RO_4) + w_3(5\ RO_4)$$

$$+ k_1(1\ I_{12} + 2\ I_8 + 3\ I_4) + k_2(1\ I_8 + 2\ I_4) + k_3(1\ I_4) \quad \ldots\ldots\ldots\ldots (7c)$$

$$O_{16} = w_1(5\ RO_{16} + 10\ RO_{12} + 8\ RO_8 + 6\ RO_4) + w_2(5\ RO_{12} + 10\ RO_8$$

$$+ 8\ RO_4) + w_3(5\ RO_8 + 10\ RO_4) + k_1(1\ I_{16} + 2\ I_{12} + 3\ I_8 + 4\ I_4)$$

$$+ k_2(1\ I_{12} + 2\ I_8 + 3\ I_4) + k_3(1\ I_8 + 2\ I_4) \quad \ldots\ldots\ldots\ldots\ldots\ldots (7d)$$

in which w and k are weighting factors to be found by least squares, RO is the runoff from local area, I is the inflow hydrograph, and the numerical values are the numeric functions.

TABLE 2.—ASSUMED NUMERIC FUNCTIONS

Time	Function for Local Inflow - N	Function for Routing - M
0	0	0
4	5	1
8	10	2
12	8	3
16	6	4
20	5	5
24	5	4
28	4	3
32	4	2
36	3	1
40	3	0
44	3	
48	3	
52	2	
56	2	
60	2	
64	2	
68	2	
72	2	
76	1	
--	--	
--	--	
144	1	
148	0	

These equations appear cumbersome, and in detailed notation for time they are unwieldy. However, they are simply expressions of an assumed condition that ordinates of the outflow hydrograph can be predicted by three weighted and lagged applications of the assumed numeric functions. While applications were chosen, any number, two or greater, could be used.

If a summation notation is used the prediction equation for outflow can be expressed more concisely as

$$O_T = w_1 \sum_{\substack{i=T\\ j=4}}^{\substack{i=T-144\\ j=144}} N_j\, RO_i + w_2 \sum_{\substack{i=T-4\\ j=4}}^{\substack{i=T-148\\ j=144}} N_j\, RO_i + w_3 \sum_{\substack{i=T-8\\ j=4}}^{\substack{i=T-152\\ j=144}} N_j\, RO_i$$

$$+ k_1 \sum_{\substack{i=T\\ j=4}}^{\substack{i=T-36\\ j=36}} M_j\, I_i + k_2 \sum_{\substack{i=T-4\\ j=4}}^{\substack{i=T-40\\ j=36}} M_j\, I_i + k_3 \sum_{\substack{i=T-8\\ j=4}}^{\substack{i=T-44\\ j=36}} M_j\, I_i \quad \ldots\ldots. \quad (8)$$

in which N is the numeric function for local area runoff, M is the numeric function for inflow, and i and j increase by 4-hr intervals.

TABLE 3.—VARIABLES FOR LEAST-SQUARES FITTING

Date	Hour	Period	RO	I	X_1	X_2	X_3	Z_1	Z_2	Z_3	0
14	8	1	.05	.04	.25			.04			.02
	12	2		.09	.50	.25		.17	.04		.09
4	4	3		.14	.40	.50	.25	.44	.17	.04	.10
	8	4		.13	.30	.40	.50	.84	.44	.17	.10
	12	5		.08	.25	.30	.40	1.32	.84	.44	.11
15	4	6		.05	.25	.25	.30	1.77	1.32	.84	.12
	8	7		.03	.20	.25	.25	2.07	1.77	1.32	.13
	12	8	.05	.03	.45	.20	.25	2.12	2.07	1.77	.14
	4	9		.02	.65	.45	.20	1.93	2.12	2.07	.13
	8	10		.02	.55	.65	.45	1.47	1.93	2.12	.11
	12	11	.10	.04	.95	.55	.65	1.25	1.47	1.93	.11
16	4	12		.12	1.40	.95	.55	1.05	1.25	1.47	.17
	8	13		.26	1.15	1.40	.95	1.19	1.05	1.25	.18
	12	14		.39	.90	1.15	1.40	1.81	1.19	1.05	.22
	4	15		.55	.80	.90	1.15	3.08	1.81	1.19	.24
	8	16		.63	.75	.80	.90	4.83	3.08	1.81	.26
	12	17		.55	.65	.75	.80	6.98	4.83	3.08	.26

The product-summation terms N RO and M I can be evaluated numerically for each ordinate, O_T, of the observed outflow hydrograph. If these product-summation terms be labeled X_1, X_2, X_3, and Z_1, Z_2, Z_3, then Eq. 8 becomes simply

$$O_T = w_1\, X_1 + w_2\, X_2 + w_3\, X_3 + k_1\, Z_1 + k_2\, Z_2 + k_3\, Z_3 \quad \ldots\ldots \quad (9)$$

and the coefficients w and k, which are linear in the equation, can be found by least squares. It should be noted that X_2 is X_1 lagged by one period, X_3 is X_1 lagged by two periods, Z_2 is Z_1 lagged by one period, and Z_3 is Z_1 lagged by two periods.

Table 3 shows the X's, Z's, and outflow, O, evaluated for the first few periods for the data shown in Fig. 8. The computations, following the form of Eq. 7, are

$X_{1-1} = (5 \times .05) = .25$

$X_{1-2} = (5 \times 0) + (10 \times .05) = .50$

$X_{1-3} = (5 \times 0) + (10 \times 0) + (8 \times .05) = .40$

$X_{1-12} = (5\times 0) + (10\times .10) + (8\times 0) + (6\times 0) + (5 \times .05) \ldots + (3 \times .05) = 1.40$

$Z_{1-1} = (1 \times .04) = .04$

$Z_{1-2} = (1 \times .09) + (2 \times .04) = .17$

$Z_{1-3} = (1 \times .14) + (2 \times .09) + (3 \times .04) = .44$

$$Z_{1-12} = (1 \times .12) + (2 \times .04) + (3 \times .02) + (4 \times .02) + (5 \times .03) + (4 \times .03) + (3 \times .05) + (2 \times .08) + (1 \times .13) = 1.05$$

Application of least squares to the complete set of data gave the following values for the weighting coefficients in Eq. 9:

$w_1 = -\ .007599$

$w_2 = \ \ .032219$

$w_3 = -\ .000890$

$k_1 = \ \ .005696$

$k_2 = \ \ .049708$

$k_3 = -\ .001069$

When the weighting coefficients, w, are multiplied by the numeric function, N, and the three resultant functions are added with proper time lag, there results a single set of distribution coefficients which is a better approximation to the unit hydrograph than the original function N. The computation is:

$$\left.\begin{aligned} U_1 &= 5\,w_1 \\ U_2 &= 10\,w_1 + 5\,w_2 \\ U_3 &= 8\,w_1 + 10\,w_2 + 5\,w_3 \\ U_4 &= 6\,w_1 + 8\,w_2 + 10\,w_3 \\ U_5 &= 5\,w_1 + 6\,w_2 + 8\,w_3 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (10)$$

The hydrograph defined by U is plotted in Fig. 9. The approximate unit hydrograph as first derived is not smooth, due principally to the discontinuous

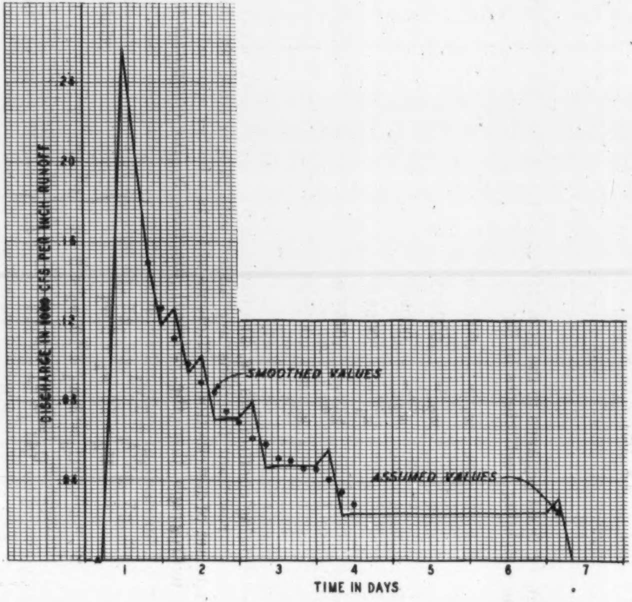

FIG. 9.—COMPUTED VALUES OF DISTRIBUTION COEFFICIENT *U*

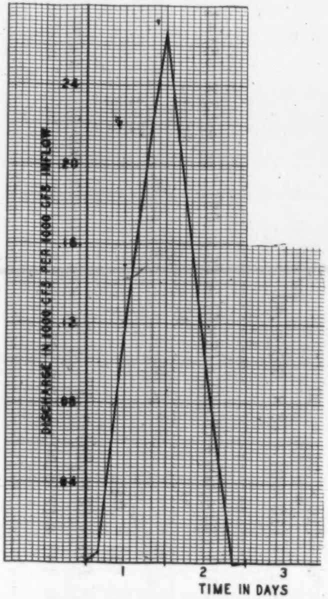

FIG. 10.— COMPUTED VALUES OF DISTRIBUTION COEFFICIENT *C*

nature of the original numeric function. To correct this, a smoothing function[8] was applied and the smooth values were also plotted in Fig. 9.

If the weighting coefficients, k, are combined with the numeric function M, in the same manner as w was combined with N, a single set of distribution coefficients result which are the routing coefficients through the reach:

$$\left.\begin{aligned} C_1 &= 1\ k_1 \\ C_2 &= 2\ k_1 + 1\ k_2 \\ C_3 &= 3\ k_1 + 2\ k_2 + 1\ k_3 \\ C_4 &= 4\ k_1 + 3\ k_2 + 2\ k_3 \\ C_5 &= 5\ k_1 + 4\ k_2 + 3\ k_3 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (11)$$

The routing coefficients, C, are plotted in Fig. 10.

After determination of the coefficients U and C they were applied to the values of runoff, RO, and inflow, I, as given in Table 3. The resulting predicted outflow hydrograph for Dentville is plotted in Fig. 8. Agreement between the observed and computed outflow is good for the major rise on November 18 and 19, but too rapid an increase is predicted for the smaller rises on November 24 and 26.

Several different procedures, which are modifications of these basic techniques, are possible. Eq. 8 could be extended to include numeric functions of preceding outflow values. Alternatively, data could be selected from several storms but limited to fixed ranges of outflow. Sets of coefficients U and C ~~could be~~ derived by separate least-squares fitting to each data set.

If more precise values of the coefficients U and C are required, some iterative procedure can be followed. In the numeric example presented, the routed hydrograph for Middle Creek plus Zion Hill could be computed after setting the sum of the coefficients C equal to unity by proportionate adjustment. The routed hydrograph of inflow could be subtracted from the Dentville outflow. The residual hydrograph of local inflow could be used to adjust the runoff values for each time period by iterative least-squares.[7]

Still another modification of the basic technique is possibe that may be of value to particular investigators. Because, for a correct unit hydrograph, the volume of runoff should be some constant, say one area-inch, this fact can be used to put restraints on the coefficients w.

The equation

$$w_1 \sum N + w_2 \sum N + w_3 \sum N = \frac{1}{\text{Area Factor}} \quad \ldots\ldots\ldots (12)$$

expresses the requirement of unit volume. This equation can be solved for w_1 as

$$w_1 = \frac{1}{(\text{A.F.}) \sum N} - w_2 - w_3 \quad \ldots\ldots\ldots\ldots\ldots (13)$$

The condition that the routing coefficients add to unity is expressed by the equation

$$k_1 \sum M + k_2 \sum M + k_3 \sum M = 1 \quad \ldots\ldots\ldots\ldots\ldots (14)$$

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[8] "Methods in Numerical Analysis," by Kaj L. Nielsen, MacMillan Co., New York, 1957, p. 291.

Solving for k_1

$$k_1 = \frac{1}{\sum M} - k_2 - k_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

The values of w_1 and k_1 can be substituted into Eq. 9 to produce the transformed prediction equation

$$O_T - \frac{X_1}{(\text{A.F.}) \sum N} - \frac{Z_1}{\sum M}$$

$$= w_2 \left(X_2 - X_1\right) + w_3 \left(X_3 - X_1\right) + k_2 \left(Z_2 - Z_1\right) + k_3 \left(Z_3 - Z_1\right) \quad \ldots (16)$$

The lefthand side of Eq. 16 can be treated as a new variable. The coefficients w_2, w_3, k_2, and k_3 can be found by least squares, using the new dependent variable and the X-differences and Z-differences as new independent variables. Coefficients w_1 and k_1 can be found from Eqs. 13 and 15 after the other coefficients are known. It should be pointed out that imposing the restraints of Eqs. 13 and 15 has reduced the degrees of freedom of the prediction equation from six in Eq. 9 to four in Eq. 16, and some loss of fitting precision will result.

The discussion presented herein suggests somewhat lengthy numerical procedures. But these computations are basically repetitious and not complex. Many or all of the steps can be programmed for electronic computation. Since the computations in many instances are reducible to matrix or vector operations, subroutines for much of the work would be available for many computers.

DAVID M. ROCKWOOD,[9] M. ASCE.—The writer wishes to thank Mr. Snyder for the contribution of his discussion on routing methods. The routing procedure he describes is similar to Tatum's method of "Successive Averages," as set forth by R. K. Linsley, Jr., F. ASCE, M. A. Kohler, F. ASCE, and J. L. H. Paulhus, F. ASCE,[10] except that the coefficients are derived by statistical weightings rather than by an arbitrary weighting system. With regard to Mr. Snyder's system, the method of statistical weighting may give unrealistic values of distribution coefficients.

This is illustrated by the values of the ordinates for the function shown on Fig. 10 of Mr. Snyder's discussion. Here, the sum of the ordinates at the end of each 4-hr period is approximately 1.35. From a strictly rational point of view, the sum of the ordinates should equal unity. The derived unit distribution graph shown on Fig. 9 for the local area inflow has a volume of about 0.31 in. rather than the theoretical value of 1 in. For this particular flood event as analyzed by Mr. Snyder, it appears that the statistical procedure "over-weighted" the channel routing coefficients, and "under-weighted" the local inflow coefficients, in order to provide a best fit of the data as a whole. It would seem particularly desirable to have the sum of the channel routing coefficients equal to unity. For the local inflow, the derived coefficients may reflect differences between assumed precipitation and loss values, and the true amounts.

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[9] Hydr. Engr., U. S. Army Engrs., Portland, Oreg.

[10] "Applied Hydrology," by R. K. Linsley, Max A. Kohler, and Joseph L. H. Paulhus, McGraw-Hill Book Co., Inc., New York, 1949.

The writer wishes to generalize briefly on streamflow routing procedures. There are almost as many and as varied routing procedures as there are hydrologists who use them. The methods range from a simple and empirical approach, to a highly complex theoretical analysis of the hydraulics of non-steady flows in open channels. In any case, it must be remembered that the basic ingredient is hydrologic data whose accuracy will always involve errors of measurement and approximations of estimates. The engineer who is confronted with the selection of a routing method should consider the problem as a whole, including the type of channel, type of study, the effect of varying backwater conditions, basic requirements of accuracy, time for solution, relative importance of results, and the resources available for computation.

In the case of the procedure outlined by the writer for the Columbia River Basin, the prime requirements were speed of operation, ability to adjust routings and forecasts on the basis of observed streamflow conditions from day-to-day, computer drum storage space limitations for storage of routing coefficients (rather than unit hydrograph ordinates), and the practical limitations that streamflow forecasts should be obtainable within 3 to 4 hr from receipt of basic data. This includes the time for processing data, adjusting routing values to observed streamflow conditions, tabulating and keypunching input values of streamflow and forecasted snowmelt, and the actual running time on the computer.

The routing procedure selected is one that is made possible by use of a medium-speed electronic digital computer. The procedure is intermediate in the scale between an entirely empirical method and a highly complex theoretical approach. The concept of the use of time of storage, T_s, is fundamental to this procedure. The fact that T_s can be computed directly for reservoirs of known storage and outlet characteristics enables the engineer to evaluate, explicitly, the inflow-outflow relationships for reservoirs. The application of the technique to evaluating storage effects in river channels by successive short reaches (or phases) of reservoir-type storage has been demonstrated. There is a physical relationship for open channels between time of storage, T_s, and the storage and flow characteristics in a given reach, which may be evaluated directly for reaches where these conditions are known by use of Eq. 5 in the original paper.

The trial and error procedure used in evaluating channel routing coefficients is actually a matter of verifying or refining values derived mathematically from generalized streamflow and channel storage data. The procedure has the additional provision for varying the time of storage (T_s) with discharge. This is a realistic approach, inasmuch as the travel time of flood waves is known to vary with the flow condition. Any function may be inserted to account for this variation. It has also been shown that successive routings through reservoir-type storage can be used to distribute runoff from rainfall or snowmelt, and by varying the routing coefficients, any desired shape of distribution may be obtained.

The procedure has been used successfully for forecasting streamflow in the Columbia River Basin during the flood season for 3 yr. For each year, forecasts were made each day for 30 to 35 consecutive days. Results have shown it to be entirely feasible, and the principal errors in verification of forecasts were due to unforeseen weather conditions. In addition to streamflow forecasting, the procedure has had the following uses: the derivation of the maximum probable flood for the Salmon River Basin, which drains about

14,100 sq miles in Central Idaho; evaluating the effect of loss of valley storage in the Priest Rapids-Wanapum reach of the Columbia River, located just upstream from the mouth of Snake River; and for general hydrologic studies of snowmelt and runoff conditions for each of twenty-one sub-basins in the Columbia Basin.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 3149

DESIGN AND CONSTRUCTION OF NAVY'S LARGEST DRYDOCK

By S. P. Zola[1] and P. M. Boothe,[2] F. ASCE

SYNOPSIS

This paper presents a technical description of the design and construction of the Navy's largest drydock at the Puget Sound Naval Shipyard, Bremerton, Washington. The ever increasing dimensions of naval ships since World War II, such as the "Super Carriers," require facilities capable of accommodating this class of vessel.

INTRODUCTION

In the old days of wooden ships and iron men the process of cleaning the ship bottom of barnacles and painting with pitch was called graving. While this name does not apply literally to the steel ships of today the title "Graving Dock" has been accepted down through the years as being synonymous with the name "Drydock." A graving drydock may be defined as a permanently fixed basin on shore into which a ship can be floated and the water removed.

Note.—Published essentially as printed here, in March, 1960, in the Journal of the Waterways and Harbors Division, as Proceedings Paper 2421. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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[2] Capt., Civ. Eng. Corps, U.S. Navy, Code D110 BuDocks Navy Dept., Washington, D. C.

The entrance, at or near the waters edge, is closed by a movable caisson or by gates.

Every ship requires at least an occasional drydocking. This may be for the purpose of normal periodic maintenance, for performance of alterations, including modernization and conversion, or for repairing battle damage. The common denominator, however, is the necessity to provide facilities that permit the underwater portions of the vessel to be exposed and available for the required work. Since the drydock, when dewatered, becomes, in effect, a repair shop it must be equipped with weight-handling equipment, compressed air for cleaning and sandblasting, power, water, oxygen, tank cleaning, and sewage-disposal facilities. In addition, a ship in drydock is completely dependent on outside sources for telephone, electricity, steam, fresh water, salt water, and sanitary sewer services to maintain living standards for the crew.

The size of naval drydocks has, by long custom, been associated with the largest class of warship that could be serviced, ranging from the small destroyer or submarine dock to the battleship dock. This latter could, until recently, accommodate the largest vessels of the Fleet. Naval strategy, tactics, and weapons development during and subsequent to World War II, however, relegated the battleship to the role of supporting fire power while emphasizing the supremacy of the carrier as the queen of the fleet. The development of carrier-based jet aircraft initiated such innovations in carrier design as the angled flight deck with its resulting overhang. To keep pace with the continued rapid and radical changes in aircraft the "Super Carrier" evolved from necessity into reality.

World War II carriers were not substantially larger than battleships, and presented no particular problems in connection with drydocking. With the advent of the Forrestal, Saratoga, Kitty Hawk, and Enterprise (nuclear) classes of carriers, however, a shortage of adequately sized drydocks for carrier repair became apparent. Although the hull of the new carriers will fit in the largest existing docks, the capability of effective underwater maintenance and repair is seriously impeded by the lack of adequate working clearances between the ship and drydock walls. Portal crane service is also hampered by the wide overhang of the angled flight deck which prevents passage of the cranes.

The completion of the Panama Canal in 1914 provided the U.S. Fleet with the capability to maneuver between the Atlantic and Pacific oceans with a minimum loss of time and thus established the two-ocean mobility concept. Subsequent ship design had been controlled by the limitation of the 100-ft wide canal locks until the development of the Forrestal. A major casualty to one of these vessels in the North Pacific could now entail a cruise more than halfway around the world due to lack of adequate drydocking facilities on the West Coast.

REQUIREMENTS

This creates an immediate requirement for repair facilities capable of accommodating the larger CVA class vessels which are already in service. This need is intensified by inclusion of future requirements for the CVA(N) (nuclear) class which are under construction.

The Bureau of Ships, which has responsibility for Naval Shipyard functions and operations as well as for design and development of new vessels, established the basic criteria for the design of the new drydock. These relate to size of dock, proximity to adjacent facilities and requirements for various utilities. The Bureau of Yards and Docks is responsible for the design and construction of the Naval Shore Establishment public works and public utilities. Such responsibility includes translation of the operational criteria into design of the finished facility.

The Puget Sound Naval Shipyard in Bremerton, Wash. has had extensive experience in construction and repair of aircraft carriers. In the post-war era this yard has been a leader in carrier conversion and was a lead yard on the design of the Midway class and of certain Essex class carrier conversions. Sound economics and judgment dictated that such experience and potential be utilized for present day needs by selection of this shipyard as the location of the Navy's largest Carrier Repair Drydock.

SITE SELECTION-OPERATIONAL CONSIDERATIONS

After selecting the geographical area one of the more significant problems faced in the drydock planning stages was that of the site selection. A number of functional, operational, and economic factors affected the final determination and were weighed and balanced. Some fifteen possible sites were investigated, not only from the operational aspect but for the purpose of evaluating comparative soil structure. Serious consideration was given to an inshore location similar to the existing docks shown on Fig. 1 but the crowded condition of available working areas restricted such possibility. An offshore location, such as sites 1A and 2 in Fig. 1, would provide laydown and working areas adjacent to the dock without infringing on existing spaces but would involve appreciably higher costs. Site 1B represents a compromise between the inshore and offshore location concepts. Since the offshore location will provide some $13\frac{1}{4}$ acres of additional valuable area for future shipyard use and will provide the contractor with a working area creating the least interference during the 3 yr of construction, the additional cost involved was considered to be warranted.

ENGINEERING ECONOMICS

During the several glacial epochs in the geological history of Sinclair Inlet, the site of the Puget Sound Naval Shipyard, deposits left by the ice movements were sorted and shifted by tidal movements resulting in variable but rather distinctive subaqueous strata. Uppermost is a layer of highly compressable soft green organic clay varying from 2 ft to 40 ft in depth, which is unsuitable for foundation material. Underlying this, in a portion of the area, is as much as 12 ft of tan to gray sandy silt, most of which must be removed. Next is a layer of compact gray silty sand and gravel grading into medium sand and gravel, and cemented fine to coarse sand and gravel having excellent bearing value.

A total of seventeen final borings supplemented the data obtained from numerous preliminary studies, which included a steel sheep pile test cell driven to a depth of 40 ft below mud line and twelve test piles driven to refusal. These studies yielded information concerning the determination of the basic

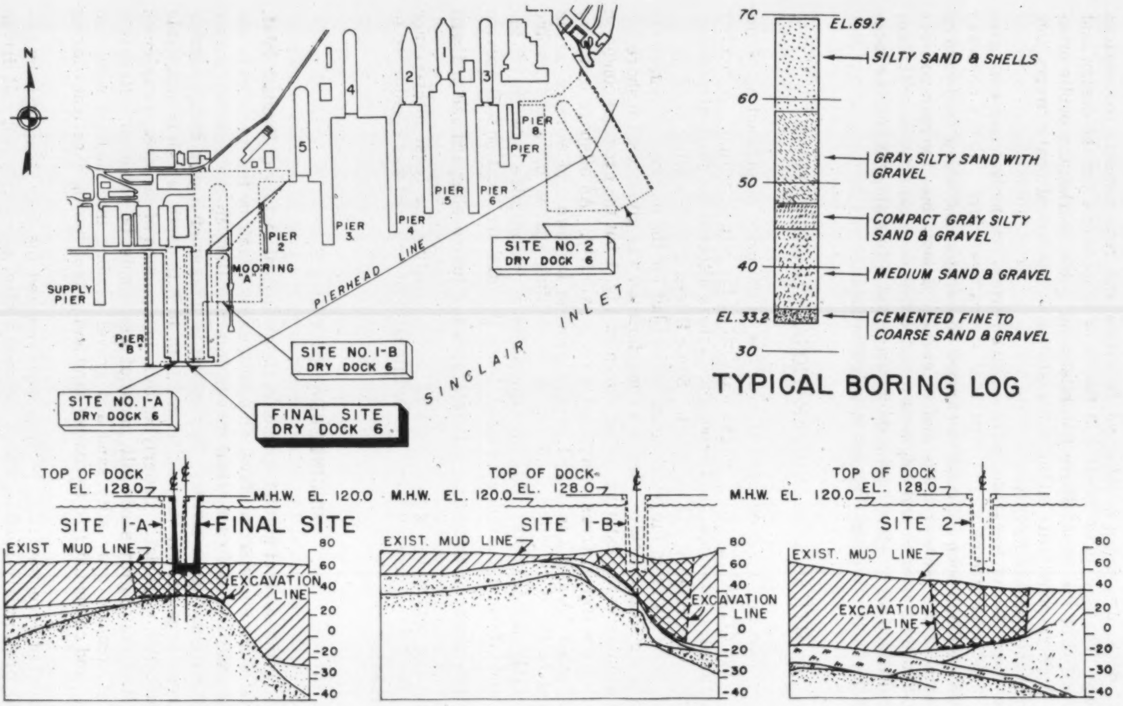

N
SUPPLY PIER
PIER "B"
PIER 2
PIER 3
PIER 4
PIER 5
PIER 6
PIER 7
PIER 8
MOORING "A"
1
2
3
4
5
PIERHEAD LINE
SINCLAIR INLET
SITE NO. 1-A DRY DOCK 6
FINAL SITE DRY DOCK 6
SITE NO. 1-B DRY DOCK 6
SITE NO. 2 DRY DOCK 6
70
EL. 69.7
SILTY SAND & SHELLS
60
GRAY SILTY SAND WITH GRAVEL
50
COMPACT GRAY SILTY SAND & GRAVEL
40
MEDIUM SAND & GRAVEL
EL. 33.2
CEMENTED FINE TO COARSE SAND & GRAVEL
30
TYPICAL BORING LOG
TOP OF DOCK EL. 128.0
M.H.W. EL. 120.0
SITE 1-A
FINAL SITE
EXIST MUD LINE
EXCAVATION LINE
SITE 1-B
EXIST. MUD LINE
SITE 2
80
60
40
20
0
-20
-30
-40

FIG. 1.—DRY DOCK SITES

drydock type, gave specific values on strength, permeability and consolidation properties of the underlying soils, and served in conjunction with the operational requirements to limit the final detailed investigations to the three sites shown in Fig. 1. Each of the final sites presented similar foundation problems, varying only in the depth to firm material and in the amount of soft overlay material to be dredged out. The final site is shown located over the high ridge of very compact gray coarse sand and gravel to minimize dredging.

The maximum tidal range is 20 ft although the average range between MLLW and MHHW is about 12 ft. The top of the mud line is approximately 54 ft below MHHW or 42 ft below MLLW. A sweep area outboard from the drydock entrance must be cleared so that no material will project above the elevation of the drydock floor or about 42 ft below MLLW.

Altogether 550,000 cu yd of unsuitable material must be removed by means of a suction dredge supplemented by clamshell and dump bottom scow. A deep water waste area one mile east of the site was used for disposal of material.

STABILITY OF DREDGED SLOPES UNDER WATER

The removal of unsuitable materials from the dredged area left side slopes as high as 40 ft in the clay stratum, requiring a slope of 1 vertical on 1.5 horizontal to provide a factor of safety greater than 2.0 to prevent material sliding into the dredged area. This comparatively steep slope is stable because of the low submerged weight of the clay and the fact that the soil strength increases in a linear ratio with the depth.

While the granular fill at the inboard end of the area has consolidated and strengthened a 15 ft underlying stratum of clay, the weight of this fill necessitates a flatter slope than the unburdened clay along the sides. On the assumption that failure could occur by rotation on a circular arc in the clay with active and passive failure wedges developing in the overlying granular material, a slope of 1 on 1.75 was determined to provide a safety factor of 1.2 submerged. To assure stability of this slope, control of ground water levels in the bank will be required when the area is dewatered.

TYPE OF DOCK

The underlying sand and gravel material was considered suitable for point bearing piles or other heavy foundation loads but pile penetration would be limited. Due to the variable thickness of the softer overlay material and the limited pile penetration where the clay stratum was thin or non-existent, foundation piling could not grip a sufficient mass of subsoil to hold down a dock floor of moderate thickness in an unrelieved dock.

To obtain the required mass of concrete to resist hydrostatic uplift in a gravity type dock, a 43 ft thick door floor slab would be required with the bottom of the slab 85 ft below MLLW. In view of the terrific construction problems and exorbitant cost involved, this type design was also discarded.

Studies were next made for a relieved type dock provided with underdrains and side drains which would dispose of sufficient ground water to reduce the hydrostatic head on the bottom and the sides to a predetermined acceptable height. Since foundation conditions were favorable, this type structure was determined to be the most economical. The design proceeded on the basis that all of the soft material overlaying the natural gravel and glacial till would be removed as having very low permeability and bearing value.

Granular material which is plentiful in the Puget Sound area will be brought in to replace the soft material and to bring the moles up to grade. This material will then become the basic foundation for the outboard end of the drydock, the mole structures, and the temporary cellular cofferdam at the entrance end of the drydock site. The inboard end of the drydock and the moles will be founded on existing sand and gravel. The granular fill must be relatively clean, graded to specifications, and capable of compaction to a degree adequate to support loads of moderate intensity imposed by the flooded drydock weight or for the more localized ship loadings when the drydock is dewatered and a ship is supported on blocking.

COFFERDAMS AND STABILITY PROBLEMS

The design adopted for the Drydock dictates that the structure be built entirely in the dry, necessitating cofferdams surrounding the drydock area. The requirement for a working area necessitates fill, (moles), to be placed along both sides of the drydock to be utilized as a basis for dams on either side of the construction area. Lines of steel sheet piles will be driven in the central portion of each dam and across the inboard end to serve as the impervious core of the dam. The earthwork and cut-off walls will remain as a functional part of the final facility. A wall of interconnected sheet pile cells is connected to the sheet piles of the dams at the outboard end to complete the cofferdam surrounding the construction site. Along the moles and the outboard portion of cellular cofferdams, the sheet piling will, be driven through the stratified glacial outwash into heterogeneous glacial till in order to minimize infiltration of ground water.

The width of the construction dams necessitates underwater fill material sloped to avoid encroaching on the drydock area or the area to be dredged for future ship berths. Granular material having the permeability and stability characteristics previously established will stand on underwater slopes of 1 on 2.75 if properly placed. Barges and flat scows, hydraulically flushed, are used to place the initial fill before pile driving is started for the sheet pile cut-off wall, thus providing support for the lower ends of the sheeting. Since the sheet piles should be driven before a substantial part of the upper portion of the dam fills are placed, precautions in placing the remaining fill are pertinent to the safety of the cut-off wall. Care must be exercised to avoid impact against the wall as well as excessive differential in heights of fill on either side.

The temporary cofferdam cells at the entrance to the dock must be filled with granular material of the same quality as used for the construction dams and other underwater fill. The material must be free draining so that excess hydrostatic pressure does not develop on the inner sheet piles and also that the weight of drained rather than submerged soil will contribute to the stability of the cells. Weep holes on the inner face of the sheet pile cells will be provided to drain the granular fill.

Limited space at the inboard end of the drydock and at the pumpwell creates stability problems requiring braced cofferdams in these areas. A steel sheet pile cofferdam at the inboard end will be necessary to permit excavation for the drydock with minimum demolition of existing buildings and to restrict ground water flow. The cofferdam will be constructed in six stages with bracing placed in sequence with the excavation against sections of the concrete floor slab. For design of the sheeting, the active and passive diagrams were determined by Coulomb's equation considering the slope of the ground surface and neglecting frictional resistance between the soil and sheet piling. A trapezoidal pressure distribution was used in designing the braces in order to allow for possible redistribution of pressures upward during excavation. The sheeting is assumed to be simply supported at each wale. Ground-water levels were assumed to decrease progressively during excavation in order to obtain reasonable active pressures and an economical design.

In excavating for the pumpwell, a braced steel sheet pile cofferdam is required to retain the fill on the interior slope of the construction dam. The cofferdam will be constructed in eight stages with bracing placed against sections of the concrete floor slab. Design assumptions were essentially the same as those for the head end cofferdam. At the lowest stages of excavation, the sheeting is assumed to cantilever about the bottom wale.

The construction area will be dewatered by means chosen by the contractor. The rate of dewatering, however, will be limited in order that ground water draw-down does not lag to an extent that will cause ravelling of the berm faces or boiling up in the bottom. Temporary construction piezometers will be provided near the inner toe of the slope of the dam so that the differential head between the ground water levels and pool level can be controlled. The contractor will also provide facilities for emergency flooding and for flooding when the drydock has been completed. These facilities must be capable of equalizing the inside and outside water levels within a period of 12 hr without damage to either temporary or permanent structures. Dewatering the construction area will require pumping 135,000,000 gallons of free water from the enclosed pool in addition to 6,000,000 gallons of drainable pore water from the construction dams. After the initial dewatering a continuing underseepage is anticipated through the sheet pile wall estimated at 20,000 to 30,000 gpm.

DRYDOCK

The drydock structure will be 1,180 ft long from the head end to outboard face, 180 ft wide at the coping, and 61 ft deep.

A typical half section of the dock is shown in Fig. 2. The dock is roughly rectangular in shape with the vertical walls at each side supported on the floor slab. In turn the walls and sloping columns support a partitioned box structure which extends the full length of the dock and serves as tunnels for the electrical and mechanical services. The relatively thin dock floor slab is 7 ft thick with edges thickened to 12 ft under the side and head end walls. The side walls vary in thickness from 12 ft 6 in. at the floor line to 2 ft 9 in. just below the utility tunnel structure. The outboard 80 ft of these walls form an entrance structure 50 ft thick for the full height to contain the caisson seats and flooding culverts. The utility tunnels are supported, as shown, on the top of the dock wall and on precast concrete columns, 22 in. square, spaced at 16-ft centers. The utility tunnel walls serve as supporting structure for the crane tracks.

The sidewalls were designed as cantilevers supported by the floor slab. The lateral design loads on the wall were assumed to be due to pressure from cohesionless granular materials, surcharge, earthquake, and a controlled

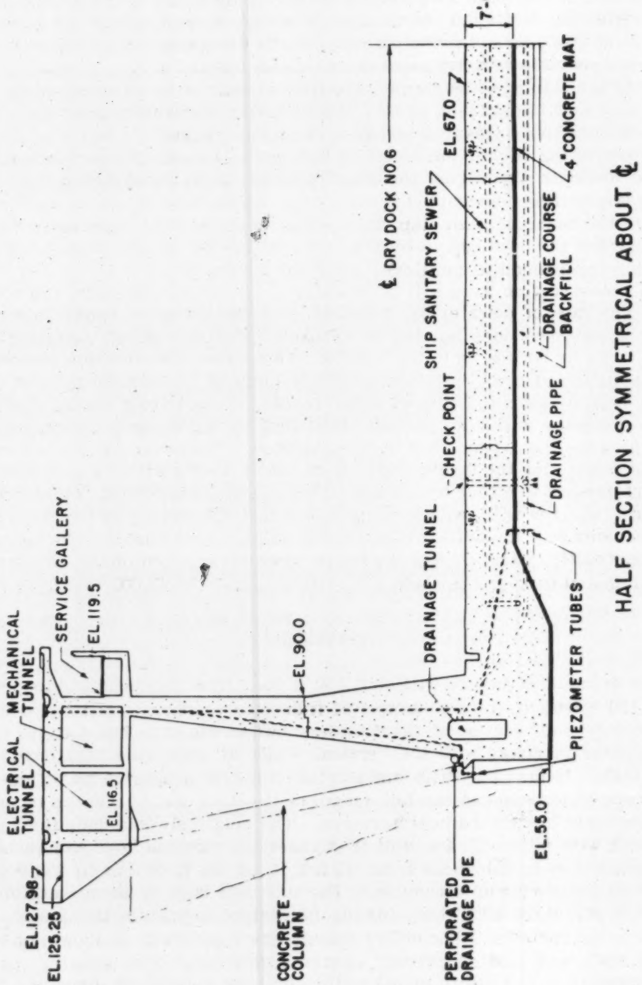

EL.127.98
EL.125.25
ELECTRICAL TUNNEL
MECHANICAL TUNNEL
SERVICE GALLERY
EL.119.5
EL.116.5
CONCRETE COLUMN
EL.90.0
PERFORATED DRAINAGE PIPE
DRAINAGE TUNNEL
EL.55.0
PIEZOMETER TUBES
CHECK POINT
DRAINAGE PIPE
SHIP SANITARY SEWER
DRAINAGE COURSE
BACKFILL
℄ DRY DOCK NO.6
EL.67.0
4" CONCRETE MAT
7'-0"
HALF SECTION SYMMETRICAL ABOUT ℄

FIG. 2.—TYPICAL SECTION

hydrostatic pressure consistent with a water level on the outside of the wall at an elevation 10 ft above the top of the floor slab. Where walls are able to deflect to the extent required to produce an active state of stress, Rankine active pressures are assumed in the fill material behind the wall. Where walls were relatively rigid and unyielding, horizontal pressures were computed, using co-efficients between those for active and "at rest" pressures. The horizontal pressures produced by surcharge were assumed to be double the Boussinesq values. A close approximation of the lateral pressures on the sidewalls are those due to a fluid weighing 32 pcf from El. 128 to El. 76, a fluid weighing 82 pcf from El. 76 to El. 70 and a fluid weighing 100 pcf below El. 70, plus horizontal pressures caused by surcharge of 75 psf down to El. 115 and 150 psf below El. 115. Concrete was specified to have a 28-day strength of 3,500 psi for the walls and floor and 6,000 psi for the precast columns supporting the service galleries. The sidewalls of the drydock will be much thinner than the massive gravity walls which have been used for most existing graving docks. Considerable reinforcing was provided in these thin walls whereas practically no reinforcing is used in the customary massive gravity walls.

The effect of these walls on the floor was considered in determining the maximum transverse moment near the wall with blocking loads of a nuclear aircraft carrier on a battleship on the drydock floor. The 12 ft thick portion of the slab under the wall was considered to be an infinitely stiff section, 25 ft long. The center portion of the floor slab, 7 ft thick, was considered to be a semi-infinite (flexible) section. The elastic foundation method of analysis was used, assuming that the distribution of pressure on the soil foundation was proportional to the deflection of the floor slab. The co-efficient of subgrade reaction was assumed to be 260 kips per sq ft per ft and maximum soil pressure computed at 5.68 kips per sq ft.

PRESSURE RELIEF SYSTEM

The basis of the design of a relieved type structure not only means that there will be insufficient dead weight incorporated in it to prevent uplift but the strength in the slab and walls will be insufficient to resist the full hydrostatic pressures. The correct design of a drainage system for obtaining the required relief and its proper construction are of the utmost importance. The system must provide a satisfactory factor of safety against uplift for normal working conditions as well as against a critical differential between the draw-down inside and outside the dock during dewatering.

The first element to consider is permeability of the backfill which is of critical importance in a relieved type of structure. The material must be so graded that the ground water draw-down approximates that of the water within the dock. This is accomplished by: (1) Granular drainage course beneath the dock floor and outside the dock walls; (2) Granular backfill, having the required permeability and stability characteristics placed between the dock wall drainage course and the moles; (3) Sheet pile cut-off walls to minimize inflow.

The drainage course will be placed against several different soils and must conform to the gradation requirements for filter stability and permeability in each situation. A gradation was established to satisfy the requirements of a filter material against all backfill and natural materials encountered except strata containing seams or lenses of very fine sand and silt. To prevent piping from any of these strata, a 1-ft layer of granular backfill will be placed beneath the drainage course in all areas where the dock is in cut.

The drainage course section outside the wall is shown in Fig. 3 and consists of a trapezoidal section approximately the shape of the active pressure wedge with a width of 5 ft at the base and 30 ft at the top. The gradation of the backfill placed between this drainage course and the moles required detailed study and balancing of the many factors involved. The material must be stable, of low compressibility and have sufficient permeability to allow the required rate of draw-down without resulting in high pumping rates while the drydock is dewatered. Conversely, the gradation should not be so restrictive as to require excessive processing and waste of the granular material locally available. Flow net studies indicated that the material should have a coefficient of permeability of approximately 0.05 ft per min.

The next basic element required is an adequate network of collector pipes under and around the drydock. After the drainage course of select material is placed under the drydock floor to permit ready percolation of seepage water, the underfloor drainage system is embedded as indicated on Fig. 3. This system is composed of a network of 8 in. perforated vitrified clay pipes with headers terminating in the sidewall drainage tunnels. The transverse headers are spaced on 48 ft centers except for the outboard end of the dock. In order to dispose of the heavier infiltration adjacent to the sheet pile cut-off wall along the dry dock entrance, the first four headers in this area are spaced at 24 ft centers. The longitudinal runs are spaced at 15 ft and $65\frac{1}{2}$ ft each side of the centerline of the drydock. At alternate intersections of the longitudinal and transverse lines, vertical risers are provided to the drydock floor in order to check the hydrostatic pressures and to permit flushing of the drainage system.

To protect the gravel drainage course from dirt and mortar and to prevent breakage of the collector pipes during construction a protective cover is required. This will consist of a membrane of impregnated paper or polyethylene covered with a 4 in. thick concrete subfloor. Exact adherence to the plans and specifications is imperative during this phase in order to assure satisfactory continued service of the drainage system. It is important both during and after laying of the collector pipe network to be alert for possible breakage and to insist upon replacement if any damage is discovered.

It will be noted that a continuous concrete cradle is provided along the outside of the dock wall to support the 12 in. vitrified clay collector pipe at the base of the wall drainage course. This pipe drains into the drainage tunnel by means of 8 in. cement asbestos branch pipes embedded in the wall at 24 ft centers. The supporting cradle will prevent dislodging the collector pipe and/or breakage of the branches into the tunnel but extreme care will be required during the backfill operation.

Flap type check valves will be provided at the tunnel ends of the drainage pipes as shown on Fig. 3. These will minimize the outflow of water into the drained ground while the drydock is being flooded and thus prevent undue disturbance of the select material drainage course and surrounding material. The flap valves will also speed up flooding of the dock by preventing flow from the tunnel into surrounding area.

The importance of the proper operation of the relief system cannot be overemphasized. If the system functions improperly a hydraulic differential

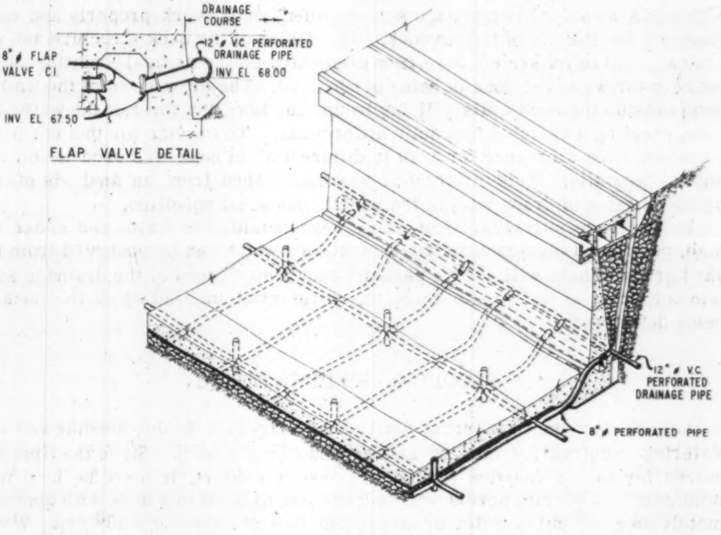

DRAINAGE COURSE
12" ø V.C. PERFORATED DRAINAGE PIPE
INV EL 68.00
8" ø FLAP VALVE C.I.
INV. EL. 67.50
FLAP VALVE DETAIL
12" ø V.C. PERFORATED DRAINAGE PIPE
8" ø PERFORATED PIPE

FIG. 3.—PRESSURE RELIEF SYSTEM

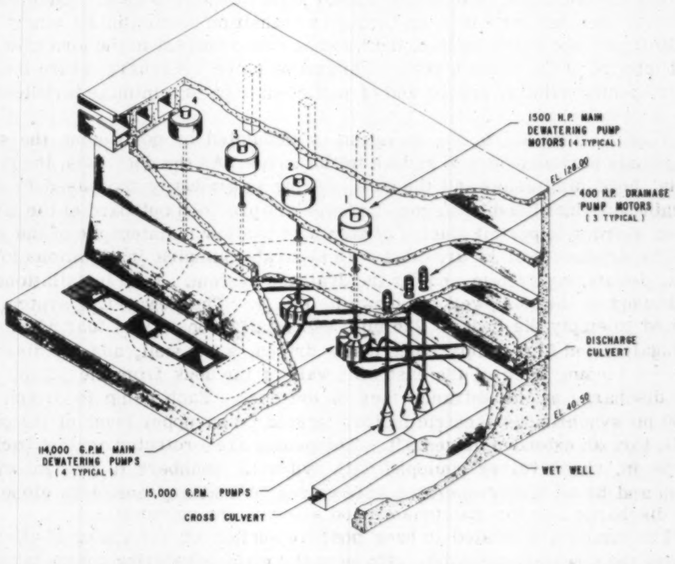

1500 H.P. MAIN DEWATERING PUMP MOTORS (4 TYPICAL)
EL 128.00
400 H.P. DRAINAGE PUMP MOTORS (3 TYPICAL)
4
3
2
1
DISCHARGE CULVERT
EL 40.50
WET WELL
114,000 G.P.M. MAIN DEWATERING PUMPS (4 TYPICAL)
15,000 G.P.M. PUMPS
CROSS CULVERT

FIG. 4.—PUMP WELL

could arise to the extent at which rupture of the bottom slab or the walls or both could occur. The drainage system, then, must work properly and continuously for the life of the drydock. The design of the dock structure and the drainage and relief system have been predicated on the critical condition which could occur while the dock is being pumped out. The draw-down of the water-level outside the dock walls will lag behind the lowering level of the water inside, creating a variable hydraulic differential. To provide for this condition a conservative allowance for a 10 ft differential in head was used in the design of the walls. This differential was established from an analysis of the probable rates of draw-down of the backfill material specified.

To facilitate surveillance of water head outside the walls and under the slab, permanent piezometers will be installed which can be observed from the yard grade. These will permit periodic recurring checks of the drainage system operation as well as to watch the differential in head while the dock is being dewatered.

FLOODING AND DEWATERING

A most important recurring function of a drydock is the flooding and dewatering required for docking and/or undocking a ship. Since the time required for such a function is lost to productive effort, it must be held to a minimum. A 90-min period will be required to flood this dock with approximately 88 x 10^6 gal of water or an average flow of almost 1 x 10^6 gpm. Water will be admitted through twin 14 ft by 11 ft inlet ports in the outboard face of each sidewall at the drydock entrance. Each port converges to a 8 ft by 8 ft culvert containing an 8 ft square motor operated sluice gate. Each pair of culverts then further converge through a transition section into a single 11 ft by 10 ft passage which leads to the flooding cross culvert in the concrete floor just inboard of the caisson seat. This cross culvert discharges thru multiple floor opening suitably spaced and of sufficient area to minimize turbulence on the dock floor.

When flooding starts the operation is modulated by positioning the sluice gates only partially open to reduce initial surge. As the dock fills, the differential head diminishes and the gate opening is gradually increased to a full opening for final flooding stages. Grooves are provided outboard of the sluices gates in order to permit placing of stop logs to allow maintenance of the gates.

The drydock will be provided with separate pumping installations for the main dewatering system and for the drainage system. Both installations will be housed in the pumpwell as shown in Fig. 4. The dewatering system, designed to empty the dock in 226 min, essentially consists of four 54 in. centrifugal mixed flow pumps electrically driven, a wet well, and the discharge culvert running back of the east side wall of the dock from the pumpwell to the discharge at the entrance face of the dock. Each pump is driven by a 1500 hp synchronous electric motor, located on an upper level of the pumpwell, thru an extended driveshaft. The pumps are protected against backflow by 54 in. check valves equipped with hydraulic snubbers to prevent violent slam and 54 in. motor-operated gate valves, permitting complete closure of the discharge line for maintenance and safety to the pumpwell.

The pumps are located to have positive suction on the intake at all times during the dewatering period. Three of the main dewatering pumps take suction from a grating covered trench in the drydock floor thru 54 in. transition elbows and 7 ft square bell mouthed conduits cast in the concrete structure. A 7 ft square, motor operated sluice gate will be installed in each intake conduit. The trench, which is 12 ft by 62 ft in plan at floor level and 4 ft deep, facilitates conveying water to any one of the three pumps during the last stage of dewatering. The spillway length provided by the perimeter of this trench will feed sufficient water at the end of the drawdown to permit one main pump to be utilized to the fullest extent with only a few inches of water on the dock floor.

The fourth pump is connected to the drainage wet well cavity thru a 54 in. transition elbow and eliptical flare mouthed conduit cast into concrete structure of the pumpwell. This intake, being lower than that of the other three main pumps, collects the last water which flows into the drainage tunnels from all parts of the drydock floor. Since a certain amount of water storage is provided in the wet well this pump will be able to operate after the other three have been taken off the line. This further postpones the time at which the drainage pumps, which are of lesser capacity, take over the final stripping.

The pressure relief drainage system, previously described, discharges into drainage tunnels in each of the two sidewalls of the dock. These tunnels drain into the wet well of the pumpwell. It is estimated that this system will collect at least 20,000 gpm which must be disposed of by continuous operation of the drainage pumps at all times that the dock is dewatered. Three drainage pumps of 15,000 gpm each, will pump the water from the wet well thru a discharge culvert into the sound. Protection against backflow is accomplished by individual check valves with hydraulic snubbers for slam protection in each pumpline as well as with 36 in. manifold gate valve and 20 in. pumpline gate valves, which allow servicing for an individual pump without shutting down the system.

CAISSON GATE

A floating caisson-type gate serves as the closure at the drydock entrance. It is a watertight, all welded steel, box-like structure approximately 176 ft long by 20 ft wide by 63 ft high with interior watertight decks and transverse bulkheads forming ballast tanks and pumping machinery compartments. The periphery of the caisson conforms to the shape and contact surfaces of the seat at the entrance. A neoprene gasket mounted on the landward face of the caisson bears against the seat forming a watertight seal. The caisson is symmetrical about the longitudinal and transverse center lines thus permitting either side to be turned toward the drydock for cleaning, painting or repairing.

The caisson is designed to resist external water pressure when the dock is empty. For this condition the caisson was considered as a rectangular plate simply supported along its edges at the sides and bottom and with a free edge at the top. The water was assumed to reach the top of the caisson. Bending and twisting moments, as well as shears and reactions, were determined by using the elastic theory for rectangular plates subject to a triangular load.

Of the horizontal girders, only the main and top decks are watertight. The main deck of the caisson is at an elevation approximately 25 ft below the top deck. The main deck serves as the operating platform for the mechanical and electrical equipment contained in the caisson.

The main framing consists of a series of vertical frames and a series of horizontal girders. A frame near each end is watertight in order to provide trim tanks. The top or weather deck is at the same elevation as the drydock coping and can be used as a roadway for pedestrian and light vehicular traffic.

The ballast tanks consist of a large center tank extending approximately three quarters of the length of the caission, and two end or trimming tanks. When closing the entrance to the flooded drydock the caisson is floated into position and then lowered into the seat by flooding these tanks. By regulating the flooding in the end tanks the caisson can be kept on an even keel during the seating operations. In removing the caisson the procedure is reversed. To provide adequate stability to the caisson when floating with the ballast tanks empty, approximately 1,860 tons of permanent concrete ballast are placed in the bottom. The amount of this ballast was partially governed by the requirement for the caisson to be raised from the sill at the lowest stage of the tide. Conversely the maximum draft of the caisson is such that it will come to rest in the seat at the highest tide that may occur. The flooding and unwatering of the caisson is controlled by the use of motor operated valves and electric driven pumps. The tanks are flooded through inlets or ports located on each side of the caisson near the bottom of the tanks. The center tank is unwatered by two 7500 gpm electric motor driven pumps and each trim tank by a 2000 gpm electric motor driven pump, all capable of operating against a total dynamic head of 30 ft. The pumps and motor operated valves are controlled from a main control panel centrally located in the machinery space above the center tank. A continuous bubbler, pneumatic-type water level indicating system with mercury manometer-type gages located at the main control panel is provided to indicate the water level in the ballast tanks and at the outside of the caisson.

Power for the pumping and also for lighting, heating and ventilation of the machinery spaces is supplied to the caisson through flexible cables from a 450 volt outlet located in the drydock service gallery near the entrance. A sound power telephone system provides intercommunication with the caisson. An electric heating and ventilating system is provided in the machinery spaces to reduce the humidity and eliminate the possibility of moisture condensation in the mechanical and electrical equipment.

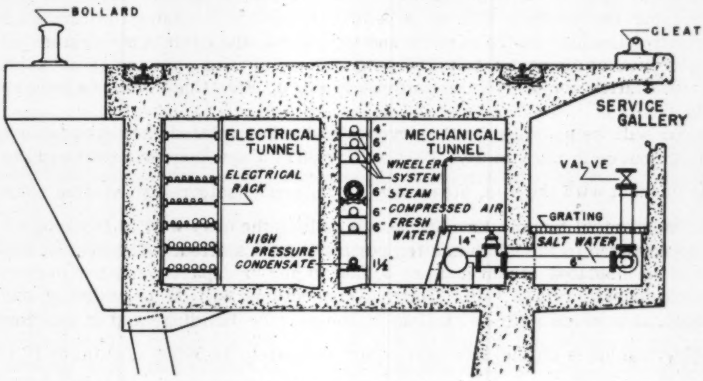
UTILITIES

The electrical and mechanical tunnels are sized nearly identically, having a common height of 8 ft 9 in. and widths of 8 ft 5 in. and 8 ft 10 in., respectively. The service gallery is continuous along both sides of the dock and averages 6 ft 8 in. high by 6 ft wide. Fig. 5 is a section through these tunnels and the service gallery showing the various pipes that will be placed in the mechanical tunnel, and the rack and cables in the electrical tunnel.

The 14 in. line for the salt water system is shown with a connection to the service gallery, at which point a grating is placed for foot traffic. This system is independent of the existing PSNS salt water system which does not cover the waterfront area at the Carrier Repair Site. A study of the existing salt water system revealed that the cost of increasing the size of the distribution main, providing additional pump capacity, and extending the main into the area affected would be prohibitive. Therefore new pumping facilities and distribution mains were established within the new drydock area, specifically designed for this purpose.

Two 3500 gpm, 200 psi pumps in the pumpwell will supply the system with an emergency flow of 7,000 gpm at 150 psi residual pressure at the remotest outlet. In addition, provisions are made on the outboard end of the moles to augment the supply by the use of fire boats. An extension to the new compressor building will be used to provide cooling water for the 5,000 cfm compressor. Pipe sizes vary from $2\frac{1}{2}$ in. to 36 in., with approximately 6,400 ft of pipe being required. Approximately 280 tons of pipe and fittings will be installed.

The existing PSNS fresh water distribution system was found to be adequate to handle the drydock requirements from an existing 12 in. main in the area. This main will be tapped opposite the west side mechanical tunnel with a 6 in. supply main, and opposite the east side tunnel with an 8 in. main. As noted in Fig. 5, one 6 in. line for the fresh water system is carried in the tunnel. Approximately 4,700 lineal ft of pipe, varying in size from $2\frac{1}{2}$ in. to 12 in., will be required. The mains will be cement-lined cast iron pipe, while the service laterals will be zinc-coated iron pipe. Approximately 150 tons of pipe and fittings will be installed.



TYPICAL SECTION

FIG. 5.—TUNNELS AND SERVICE GALLERY

The requirement for compressed air at the Carrier Repair Site is difficult to determine. Changing ship design, variations in requirements in ships of the same class, and differences in repair operations make an accurate estimate of compressed air loads virtually impossible. Consideration was given to provide compressed air to care for the maximum demand load that could occur under the worst conditions but, as might be expected, this resulted in a highly over-designed system with excessively large mains and excessive costs. The final requirements were established after a study of the existing drydock loads and comparison with estimated requirements in the new dock for a large carrier. Since the existing air generation equipment was found to be loaded at approximately 95% of capacity and the existing distribution mains in the project area were of insufficient size to handle the expected load, it was found necessary to install a new 5,000 cfm at 100 psi compressor in a new building near the repair site. A 10 in. steel main will be run under ground to the northwest corner of the dock from which the 6 in. line noted in Fig. 5 will loop through the mechanical tunnel. Altogether some 4,800 lineal ft of pipe varying in size from 3 in. to 10 in. and weighing approximately 54 tons will be required for the compressed air system.

As was the case with compressed air, the requirements for oxygen in the new Carrier Repair Site are extremely difficult to determine. The quantities required and sizing of the lines was determined in the same manner from studies of past requirements at Shipyard drydocks. The existing PSNS oxygen generation plant has sufficient capacity to handle the increase in load for the new facility and an existing distribution main in the vicinity will be tapped with a 4 in. extra strong wrought iron pipe with welded joints. This pipe will extend underground to the northeast corner of the drydock at which point the line will be teed and a 3 in. main carried down each side of the drydock, buried in the fill. It will be noted that the oxygen line is not shown in the tunnel since the hazards involved in handling oxygen required independent burial of the line. Service for the drydock will be provided by outlets crossing the tunnel and terminating in a box on the surface and adjacent to the curb. This system will carry oxygen at approximately 100 psig for use abroad ship and at dockside. The installation complies with National Board of Fire Underwriters requirements.

Steam will be provided to the drydock and other related facilities from an existing source in the vicinity. This will require a new concrete duct and piping to connect with the 6 in. steam and $1\frac{1}{2}$ in. condensate return system shown in the mechanical tunnel. Existing service along the quay wall to Mooring "A" will be rerouted to the north and temporary lines installed. The system consists of an insulated steam main of 80,000 lb per hr capacity and a high pressure condensate return line. Service connections will be provided at each mechanical service gallery station, in the service building, and at the three Wheeler manholes on the East mole. Approximately 10,300 ft of $\frac{1}{2}$ in. to 12 in. diameter pipe will be required, with an estimated total weight of 125 tons.

A process known as the Wheeler System is used to loosen oil sludge and remove it from shipboard storage tanks while the vessel is in drydock. To accomplish this operation, fresh water is taken into the Wheeler units, which are located on skids, and heated to near boiling with steam. The heated water is pumped through the high-pressure Wheeler line into the ship's oil tanks where it loosens the oil sludge and converts it to a froth. The dirty water and froth are removed through the suction line and are pumped into a floating tank at the seaward end of the dock. The Wheeler system valve bank will thus connect to the fresh water, steam, and compressed air as well as the 6 in. suction and discharge, and 4 in. high-pressure Wheeler system lines shown on Fig. 5. A total of 3,500 ft of 4 in. to 6 in. pipe is required with an estimated total weight of about 45 tons.

The ships sanitary sewer system consists of ten transverse 6 in. cast iron lateral sewer lines located at 96 ft centers in the floor of the dock. Provisions will be made for six connections to each lateral, except for the two inboard laterals which will provide for five connections each. The sewage will drain to a sewage wet well in the pumpwell, where pumps will transport it to a lift station in the Shipyard Interceptor Sewer System. Two 300 gpm at 90 ft head suction pumps will be provided, each pump capable of passing 3 in. solids. The pumps will be driven by a 20 hp electric, open. drip-proof motor. The system is designed to serve a population of 3,100 men and contains 3,500 lineal ft of pipe, varying from 6 to 10 in. in diameter weighing approximately 80 tons.

Included in the electrical tunnel are telephone, fire alarm, and secondary services. Primary services are carried in ducts and manholes to the substations and pumpwell. Cables will be carried on racks as shown in Fig. 5 and terminated in boxes at the service stations which will be provided at approximately 150 ft intervals in the service gallery.

ELECTRICAL SYSTEM

Electrical energy is available in the Shipyard from three sources. The Bonneville Power Administration can deliver 27,000 kva through two transformer banks from a 115 kv supply to the PSNS 11.5 kv feeder system. The Puget Sound Power and Light Company can deliver 15,000 kva through a transformer bank from their 69 kv system. The Central Power Plant of the Shipyard can produce 18,000 kva of emergency power. Electrical energy is distributed underground by an 11.5 kv, three-phase system to substations scattered throughout the shipyard so that any one or all three sources of power can supply any load.

The primary feeder system to the Carrier Repair Site, shown in Fig. 6 is arranged to be flexible and to provide a maximum of service. As such it follows the pattern of the existing loop system which reaches any distribution substation from at least two main stations. Primary 11,500 volt power is received at substations 83, 84 and 85, and the compressor building to form a new loop following the established shipyard pattern. Each cable run terminates with a circuit breaker having 250,000 kva interrupting capacity and is protected by a pilot wire relay system. Connections to the Shipyard Primary 11,500 volt network will be made at existing Substation F, northwest of Drydock No. 4; and in Manhole No. 191 south of the new Compressor Building.

Transformer capacity of approximately 21,250 kva will be installed as follows:

Substation No. 83 - 9750 kva
Substation No. 84 - 3000 kva
Pumpwell Substation - 7500 kva
Compressor House - 1000 kva

The primary network cables shown by dash-dot line in Fig. 6 will be run through underground ducts and manholes between terminal points. The secondary cables at the drydock will be run between the substations and service galleries on cable racks installed in the electrical tunnel as previously shown. There will be seven shore to ship connections on each side of the dry dock, each providing a variety of voltages.

An approximation of the size of the installation can be gained by the following quantities required:

12 miles of 500,000 cm., 15 kv primary cable
8 miles of three-conductor cable

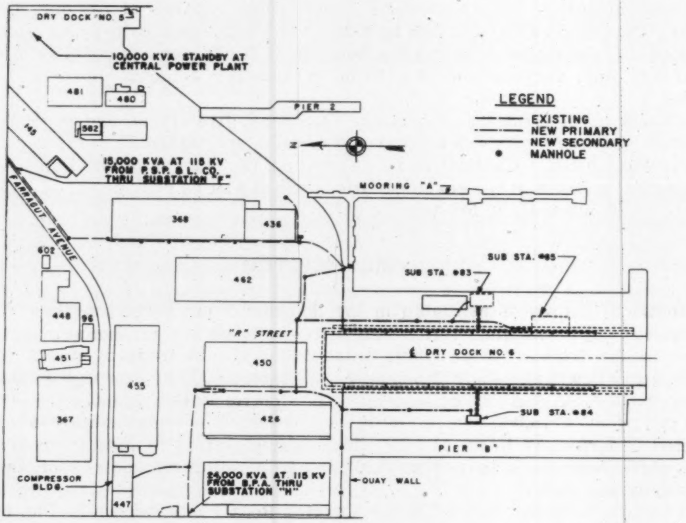

DRY DOCK NO. 5
10,000 KVA STANDBY AT CENTRAL POWER PLANT
481
480
582
145
PIER 2
LEGEND
EXISTING
NEW PRIMARY
NEW SECONDARY
MANHOLE
N
15,000 KVA AT 115 KV FROM P.S.P. & L. CO. THRU SUBSTATION "F"
FARRAGUT AVENUE
MOORING "A"
368
436
602
462
SUB STA. #85
SUB STA. #83
448
96
"R" STREET
DRY DOCK NO. 6
451
450
455
367
426
SUB STA. #84
PIER "B"
COMPRESSOR BLDG.
24,000 KVA AT 115 KV FROM B.P.A. THRU SUBSTATION "H"
QUAY WALL
447

FIG. 6.—ELECTRICAL POWER

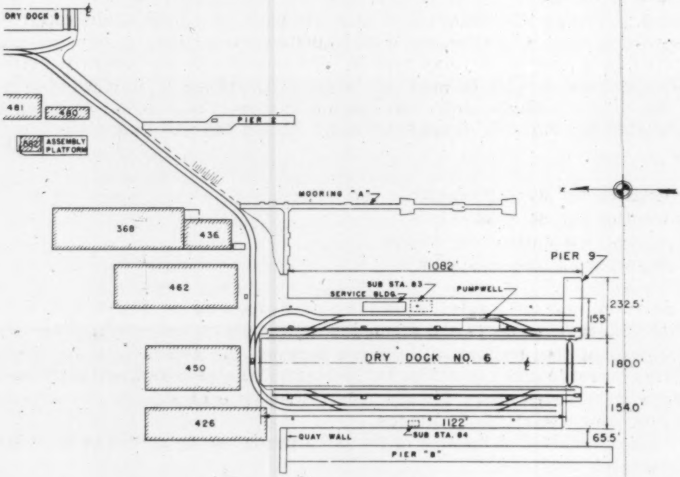

DRY DOCK 5
481
480
PIER 2
582
ASSEMBLY PLATFORM
N
MOORING "A"
368
436
462
PIER 9
1082'
SUB STA. 83
SERVICE BLDG.
PUMPWELL
232.5'
155'
DRY DOCK NO. 6
1800'
450
1540'
426
1122'
QUAY WALL
SUB STA. 84
65.5'
PIER "B"

FIG. 7.—DRY DOCK NO. 6

12 miles of miscellaneous cables
10 miles of underground duct
2 miles of underground steel conduit
1/2 mile of cable tunnel
Nine electrical manholes
Six steel floodlight towers, approximately 100 ft high

The telephone service will consist of an extension of the Shipyard standard system connected to the Yard exchange in Building 78 through a 202 position main frame and approximately 4,600 ft of 152 pair telephone cable in the main run. Cables with lesser number of pairs will be routed to various locations at the Carrier Repair Site to provide telephone service at needed points, including connections to ships in drydock. In addition there will be a self-contained sound powered telephone circuit to facilitate the docking of ships. The phone jacks will be installed along the drydock sides at appropriate locations.

The existing fire alarm system at the Shipyard is a service loop circuit with coded wheels in the alarm boxes. It will be extended to the drydock area and alarm boxes will be installed at selected locations on the moles and in substations. In each of the fourteen electric service galleries provisions for extension of this fire alarm system to the ship in dock can be made by temporary cable connections. One existing fire alarm box will be relocated, and eight new master and two auxiliary boxes will be installed. Connection will be made to the Shipyard fire alarm system in the manhole west of Building 450.

MISCELLANEOUS STRUCTURES

Fig. 7 is a plan view of the completed first increment of the Carrier Repair Site showing the crane tracks in place, the cassion seated in the inner seat which gives a clear inside drydock length of 1,122 ft. Crane track connections to the existing system near drydock 5 are indicated as well as Building 450 and 480 which require shortening to provide space for the crane tracks.

For the first increment of construction only a relatively small portion of Pier 9 is to be built at the southerly end of the east mole as shown in Fig. 7. The remainder of the pier will be constructed as a future increment. The same general principles regarding design and pile driving, however, were used for the pier stub and the final pier construction. The bearing and batter piles will be of pre-stressed concrete with conventional cast-in-place reinforced concrete pier decks. Timber piles backed up by rubber bumpers will be used for fenders. From the study of the results of test borings, test piles, and records of tests and bearing piles of existing structures, it was concluded that piles for the support of the piers would be essentially end bearing. A minimum penetration line has been established, both for test piles and foundation piles for Pier 9. This is required, even tho side friction in the fill may give the required 80 ton bearing value above this line, since skin friction in the fill may ultimately be decreased by settlement. The Contractor is allowed to jet to within 3 ft of minimum required penetration since this will tend to eliminate much of the unreliable skin friction. A design loading of 600 lb per sq ft is used for pier decks while bulkheads are designed with a surcharge of 300 lb per sq ft, assumed to act over an infinite length of wall.

The crane track installation on the concrete structure as shown in Fig. 5, is original for this project in such respects as: (1) The anchor bolts do not extend into the main structural concrete; (2) The slots in the main concrete to contain the rail and casing concrete slope inward at the sides to increase shear resistance and to provide for positive, inverted arch action for resisting uplift due to flexible foundation action; and (3) Non-shrink producing admixture is specified for the encasing concrete to minimize any slight movements which might develop at the sides from the flexible foundation action. For track work on other than concrete, the rails and fittings will be placed on treated wood ties to allow for reballasting in the event of settlement of the fill.

A rather extensive program of demolition and modification of existing structures, utilities, pavements and railroads has been necessary to provide space for the drydock and related facilities. The demolition will include removal of seven large buildings, fuel oil loading and transfer facilities including a 20 ft by 532 ft fuel oil pier, four old shipbuilding ways, 570 lineal ft of concrete sea wall and a portion of the existing underground electrical system containing lighting circuits, fire alarm circuit, 26 pair and 101 pair telephone cables as well as primary power.

The original design criteria required the use of elevators or escalators or a combination of both to carry workmen and materials to and from the drydock floor. The use of escalators was abandoned because of the cost and difficulty of sealing the units from the corrosive effects of salt water during the flooded period. A design was prepared for an elevator unit in a shaft on the east side near the head end of the drydock, set back approximately 100 ft from the coping line. A tunnel with floor elevation flush with the drydock floor would connect the lower end of the shaft with the drydock. For economic reasons however, this design was eliminated and a pit will be provided in the northeast corner of the drydock for possible future use. This pit will be filled with a lean concrete for the present time.

CONSTRUCTION REQUIREMENTS

The usual custom in Navy construction work is to give the general contractor the widest possible latitude in construction methods. In a significant departure from convention, however, the design of the Carrier Repair Site requires a specific sequence of construction operations and exact adherence to the plans and specifications to assure satisfactory functioning of the facility. This mandatory construction sequence, which is incorporated in the plans and specifications, consists of the following four phases:

Phase 1 - Dredging and Initial Fill
Phase 2 - Cofferdam Construction and Embankment
Phase 3 - Site Preparation
Phase 4 - Drydock Construction

The initial requirements of Phase 1 include the dredging and a portion of the demolition previously described. The fuel pier previously listed for removal was located in the area to be dredged, thus requiring early disposal. A prerequisite to demolition of the Fuel Pier, however was construction of a replacement fuel oil facility to preclude interference with Shipyard operations.

Removal of unsatisfactory foundation material by dredging has been previously covered. The dredged material was pumped through a 6,000 ft discharge line to a disposal area in a deep portion of Sinclair Inlet. A thought-provoking problem arose when it was discovered that conventional sweeping methods, conducted after dredging to grade was accomplished, were unsuccessful in removing a boundary layer of loose silt. This layer, up to 3 ft in thickness, was in a flocculated semi-liquid state and was obviously unsatisfactory as a base for permanent fill. Although there was reasonable speculation that this layer could be displaced laterally as a mud wave by filling operations, this occurrence was by no means assured and the possibility of a plane of low shear strength and high compressibility remained a problem. The final solution was placement of a $\frac{3}{4}$ in. to 3 in. gravel blanket to assure foundation-to-fill interlock and to provide the required shear strength. Care was taken to restrict the gravel blanket to the area inboard of the cut-off wall to prevent a seepage channel of high permeability.

Fig. 8 shows the initial construction phase for the drydock after the dredging has been completed. It will consist of filling the dredged area to El. 70 with a select material barged in from a borrow pit constructed especially for this job about 3 miles across Sinclair Inlet from the Shipyard. At the option of the contractor, the fill may be humped where the sheet pile cut-off walls will be driven. Approximately 451,000 cu yd of fill will be deposited in this phase with a minimum placed in the dry dock area since it will later have to be removed.

Fig. 9 shows the second phase of drydock construction. The first operation during this phase has been to drive the sheet pile cut-off wall along each side of the dock and construct an eleven cell sheet pile cellular cofferdam around the outboard end of the dock. The contractor started phase 2 operations in a portion of the area where the dredging and initial backfill had been completed.

The area above the broken line in Sections A-A and B-B shows where additional backfill is to be placed. As shown in Section A-A, the fill will be brought to El. 116, or 7 ft above MLLW, and will be sloped $2\frac{3}{4}$ to 1. Fill work is being accomplished by hydraulic washing methods from barges. Approximately 605,000 cu yd of fill will be required.

A rock facing for tidal protection will be placed on the fill slope at the outboard side of the sheet piling. The width of the berm inboard of the cut-off wall will be 30 ft. As will be noted in Section B-B, the outboard side of the cellular cofferdam is driven into the subgrade, whereas the inboard piling are only driven into the first stage of the selected fill to predetermined cut-off grades.

The top of the piles in the cut-off walls and cellular cofferdams will be about one foot higher than the elevation of extreme high water for this area to preclude the possibility of topping and attendant possible loss of the structure. Pile lengths will vary from 95 ft to 99 ft for the outer face and 63 ft for the inner face. The cellular cofferdam will be removed in a later phase of the construction.

Site preparation, or construction phase 3, is shown in Fig. 10 and consists of dewatering the site, trimming the side slopes, necessary site excavation in the dry and the construction of the head-end and pumpwell cofferdams. As previously noted the dewatering procedures are the responsibility of the contractor but are subject to the approval of the Officer in Charge of Construction. The contractor will also make provision for a temporary flooding system of such size and capacity as to flood the drydock to a balanced water level

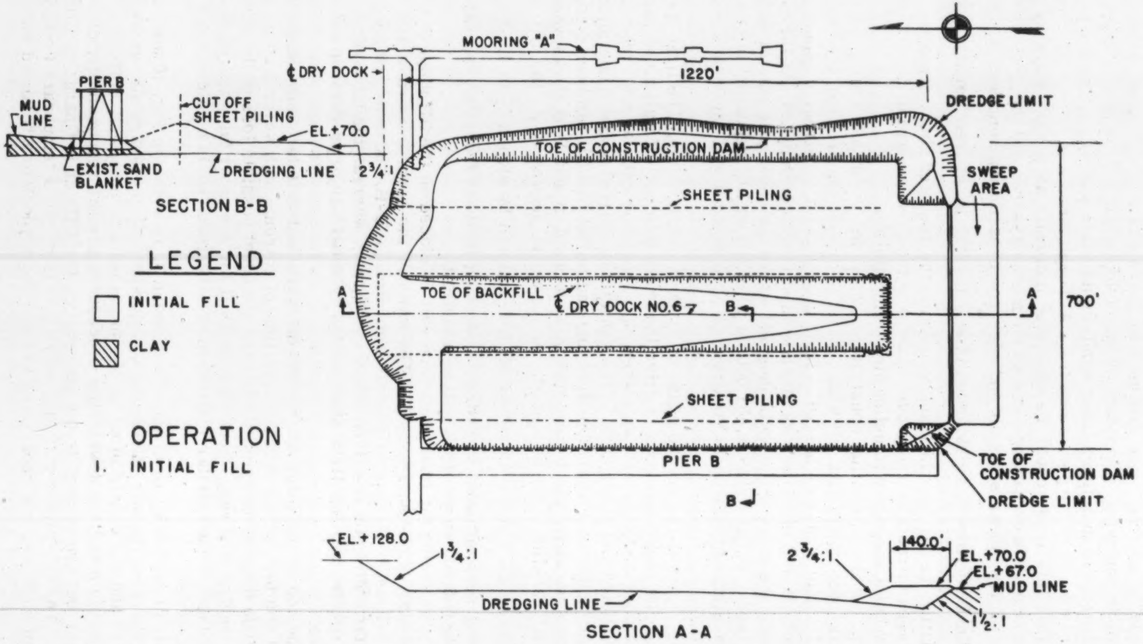

MOORING "A"
℄ DRY DOCK
1220'
DREDGE LIMIT
TOE OF CONSTRUCTION DAM
SWEEP AREA
SHEET PILING
A
TOE OF BACKFILL
℄ DRY DOCK NO. 6
B
A
700'
SHEET PILING
PIER B
TOE OF CONSTRUCTION DAM
DREDGE LIMIT
B
MUD LINE
PIER B
EXIST. SAND BLANKET
CUT OFF SHEET PILING
EL. +70.0
DREDGING LINE
2 3/4:1
SECTION B-B
LEGEND
INITIAL FILL
CLAY
OPERATION
1. INITIAL FILL
EL. +128.0
1 3/4:1
2 3/4:1
140.0'
EL. +70.0
EL. +67.0
MUD LINE
1 1/2:1
DREDGING LINE
SECTION A-A

FIG. 8.—INITIAL FILL (PHASE 1B)

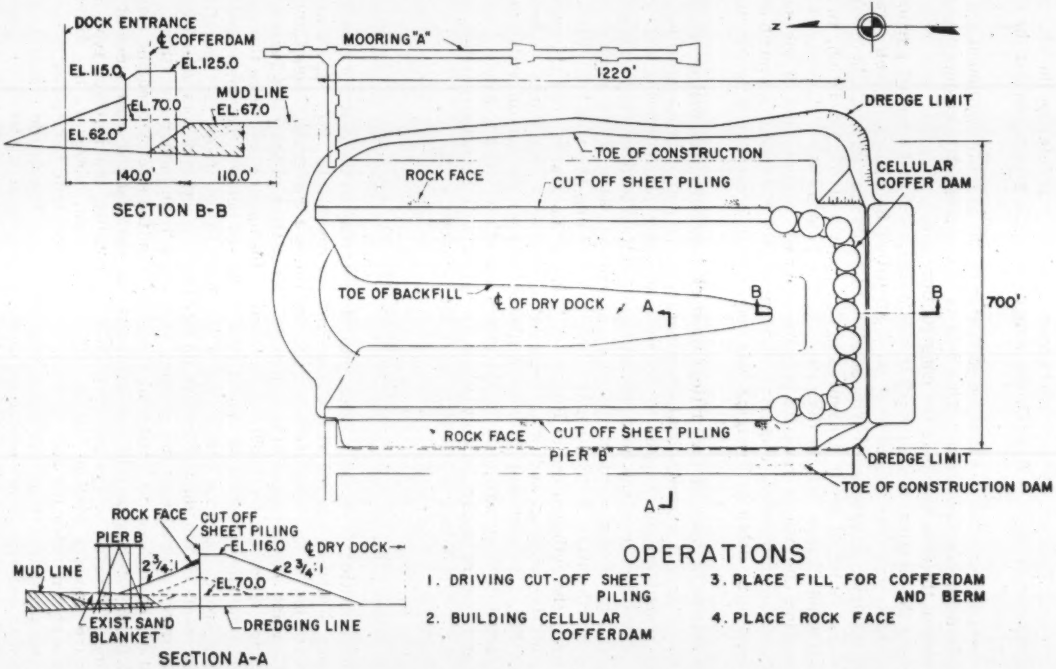

DOCK ENTRANCE
℄ COFFERDAM
EL.115.0
EL.125.0
MUD LINE
EL.70.0
EL.67.0
EL.62.0
140.0'
110.0'
SECTION B-B
MOORING "A"
1220'
Z
DREDGE LIMIT
TOE OF CONSTRUCTION
ROCK FACE
CUT OFF SHEET PILING
CELLULAR COFFER DAM
TOE OF BACKFILL
℄ OF DRY DOCK
A
B
B
700'
ROCK FACE
CUT OFF SHEET PILING
PIER "B"
DREDGE LIMIT
TOE OF CONSTRUCTION DAM
A
ROCK FACE
CUT OFF SHEET PILING
PIER B
EL.116.0
℄ DRY DOCK
MUD LINE
2¾:1
2¾:1
EL.70.0
EXIST. SAND BLANKET
DREDGING LINE
SECTION A-A
OPERATIONS
1. DRIVING CUT-OFF SHEET PILING
2. BUILDING CELLULAR COFFERDAM
3. PLACE FILL FOR COFFERDAM AND BERM
4. PLACE ROCK FACE

FIG. 9.—COFFERDAM AND BERM FILL (PHASE 2)

within a 12 hr period. This provision is made for the event of an emergency wherein damage to the cofferdam and existing structures might result if the area were not flooded. The ground water level after dewatering the site is to be maintained at least 2 ft below the subgrade of the excavation for the dock and pumpwell.

A construction road is shown on each side of the site for equipment access to the drydock subgrade area. This will be accomplished in conjunction with trimming of the side slopes of the construction dam.

During the original design work it was anticipated that a small portion of the drydock structure would rest on a stratified foundation of well graded gravel material interspersed with some hard sandy clay lenses. In order to insure the relief of uplift pressures in this portion of the foundation, twenty vertical sand drains, 2 ft in diameter and 15 ft deep, were planned. Field construction operations revealed a more extensive distribution of the sandy clay than was anticipated, requiring removal of this material. As much as 20 ft of additional depth of dredging was required in some areas to provide a satisfactory foundation material.

Excavation of most of the critical drydock subgrade area will be made in the dry. This will permit visual examination of the subgrade material and allow for possible field changes to insure an adequate foundation for the structure. Construction of the head end and pumpwell cofferdams in the locations shown in Fig. 10 will complete the site preparation phase of the construction.

The fourth phase of construction involves building the concrete drydock, pumpwell, the outboard end of Pier 9, utilities, crane tracks, paving, substations, utility building, and other miscellaneous items. As shown in Section B-B of Fig. 11, additional fill will be placed between the cut-off wall and the constructed drydock wall to bring the moles and other areas to grade. Three floodlight towers approximately 100 ft in height will be constructed on each mole to provide 3-foot candle lighting in the lay-down area.

CONCLUSIONS

The transition from the wooden ship of yesterday to the Navy of tomorrow provides a history replete with change and improvement. Fig. 12, showing a cross section of the completed project containing a Forrestal Class Carrier, could furnish another page in this history. Completion of the First Increment, now under construction (1960), is pictured in the upper view while the entire project, including a complete repair pier, is shown below. Laydown area and working space shown on either side of the dock result from the operational requirements of the shipyard which dictated the selection of this offshore site in preference to reducing the existing yard area by an inshore location. The extent of the organic clay material removed from the site by dredging may be noted by the overlay of this material shown at the east and west sides of the drawing. Dwarfed by the massive CVA it contains, the relieved type dock structure appears almost fragile. It is protected, however, by drainage courses shown under the deck and along the sidewalls to insure functioning of the vital pressure relief system. Representative foundation conditions are noted with one side shown embedded in the firm glacial till while the other is supported on select backfill. Other innovations and items of import in the design and construction of the Navy's largest drydock are the unique features of the flooding and dewatering systems, the prototype caisson and the prearranged compulsory phasing of the construction operations.

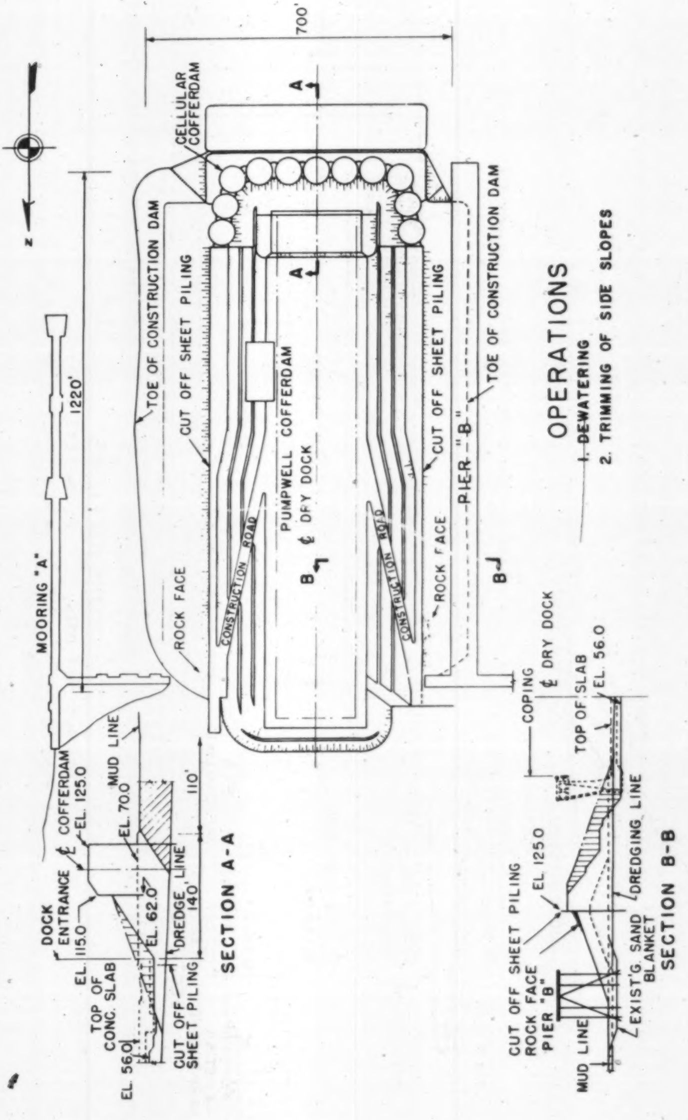

700'
1220'
N
MOORING "A"
CELLULAR COFFERDAM
TOE OF CONSTRUCTION DAM
CUT OFF SHEET PILING
PUMPWELL COFFERDAM
℄ DRY DOCK
CONSTRUCTION ROAD
CONSTRUCTION ROAD
ROCK FACE
ROCK FACE
CUT OFF SHEET PILING
TOE OF CONSTRUCTION DAM
PIER "B"
A
A
B
B
OPERATIONS
1. DEWATERING
2. TRIMMING OF SIDE SLOPES
DOCK ENTRANCE
EL. 115.0
℄ COFFERDAM
EL. 125.0
MUD LINE
EL. 70.0
TOP OF CONC. SLAB
EL. 62.0
EL. 56.0
DREDGE LINE
CUT OFF SHEET PILING
140'
110'
SECTION A-A
COPING
℄ DRY DOCK
TOP OF SLAB
EL. 56.0
CUT OFF SHEET PILING
EL. 125.0
ROCK FACE
PIER "B"
MUD LINE
EXIST'G. SAND BLANKET
DREDGING LINE
DREDGING LINE
SECTION B-B

FIG. 10.—SITE PREPARATION (PHASE 3)

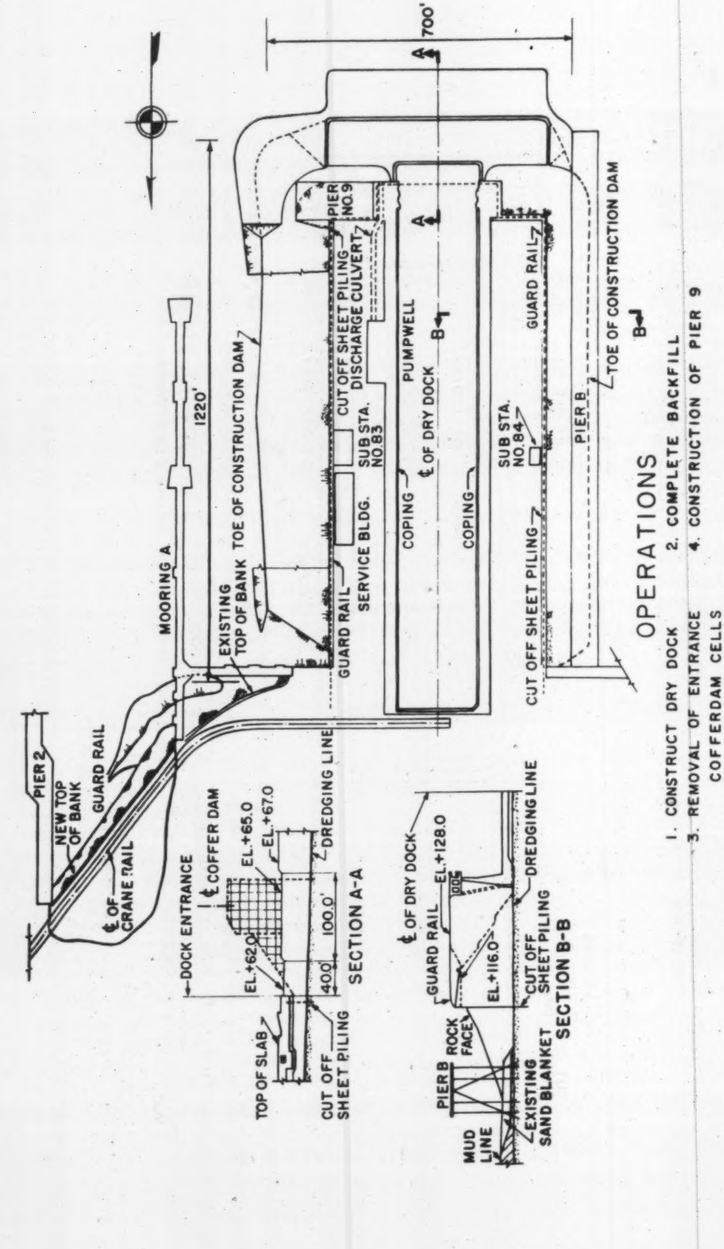

700'
A
1220'
PIER NO.9
CUT OFF SHEET PILING
DISCHARGE CULVERT
PUMPWELL
GUARD RAIL
TOE OF CONSTRUCTION DAM
B
SUB STA. NO.83
SUB STA. NO.84
PIER B
℄ OF DRY DOCK
COPING
COPING
SERVICE BLDG.
TOE OF CONSTRUCTION DAM
EXISTING TOP OF BANK
MOORING A
GUARD RAIL
CUT OFF SHEET PILING
PIER 2
NEW TOP OF BANK
GUARD RAIL
℄ OF CRANE RAIL
DREDGING LINE
DOCK ENTRANCE
℄ COFFER DAM
EL.+65.0
EL.+67.0
EL.+62.0
DREDGING LINE
100.0'
40.0'
TOP OF SLAB
CUT OFF SHEET PILING
SECTION A-A
℄ OF DRY DOCK
EL.+128.0
GUARD RAIL
DREDGING LINE
EL.+116.0
CUT OFF SHEET PILING
ROCK FACE
PIER B
MUD LINE
EXISTING SAND BLANKET
SECTION B-B
OPERATIONS
1. CONSTRUCT DRY DOCK
2. COMPLETE BACKFILL
3. REMOVAL OF ENTRANCE COFFERDAM CELLS
4. CONSTRUCTION OF PIER 9

FIG. 11.—DRY DOCK IN PLACE - INCREMENT I (PHASE 4)

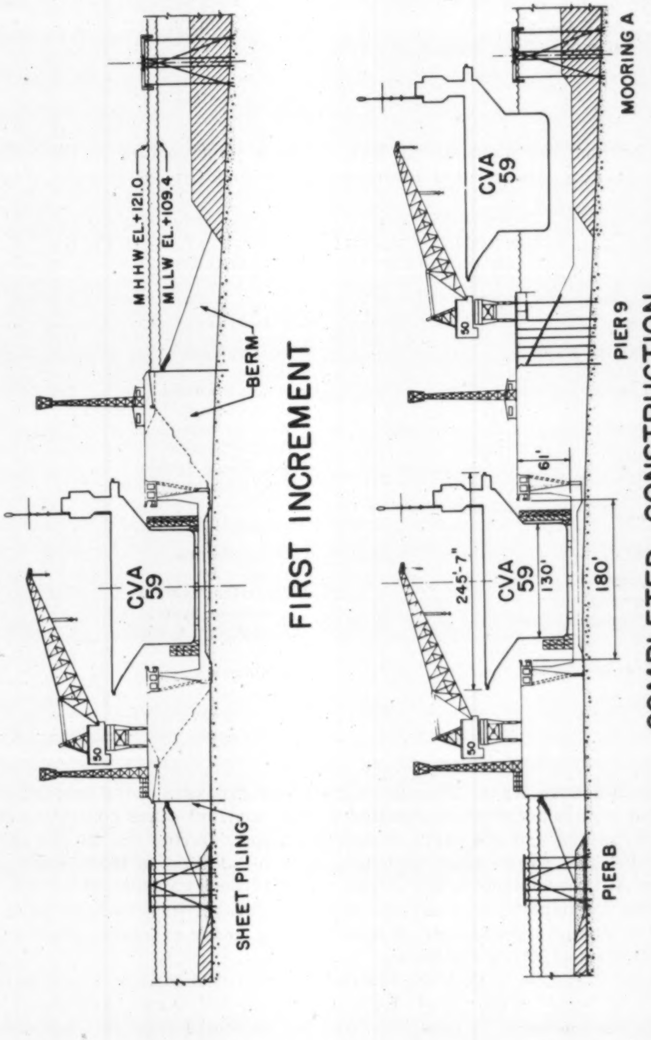

MHHW EL.+121.0
MLLW EL.+109.4
BERM
SHEET PILING
CVA 59
50
FIRST INCREMENT
CVA 59
50
245'-7"
130'
180'
61'
PIER B
PIER 9
MOORING A
COMPLETED CONSTRUCTION

FIG. 12.—CARRIER REPAIR SITE

The relative workload for this project by construction categories is listed in Table 1. This shows a preponderance constituting over 75% of the job to be in the dock structure and associated marine work. A further concept of the scope of the project may be gained by review of a summary of the basic material quantities involved as shown in Table 2. This table, listing the quantities of major material items required for construction provides a comprehensive estimate of the magnitude of the project.

ACKNOWLEDGMENTS

The project was planned and designed by a joint venture of the firms of Moran, Proctor, Mueser and Rutledge of New York City, N.Y. and Carey and Kramer of Seattle, Wash. The planning and design phases of the project, which required 3 yr to accomplish resulted in final contract plans consisting of 303 sheets drawings and 364 pages of specifications. During the 60 day bidding period 197 sets were issued to contractors in addition to those supplied to various AGC headquarters and others. The drydock caisson, which will be a prototype for future super-carrier drydocks at other locations, is being designed by Foster and Cafarelli of New York City under a contract administered by the Bureau of Yards and Docks.

TABLE 1.—APPROXIMATE PERCENTAGES OF TOTAL ESTIMATED COST OF DRY DOCK NO. 6, BY CATEGORIES

Demolition Shipbuilding Ways Fuel Oil Pier Buildings	0.6%	Mechanical Piping in Docks Pumps Steam System Compressed Air System Wheeler System Oxygen System Salt Water System Fresh Water System	5.4%
Marine Work Dredging Cofferdam & Berm Backfill Drainage	25.0%		
Structural Concrete Reinforcing Misc. Structural Steel	50.2%	Electrical Pump Motors Substations Electrical System Telephone System Fire Alarm System	8.9%
Crane Tracks	4.1%	Miscellaneous	4.4%
Pavements	1.4%	Total	100.0%

Captain S.P. Zola, CEC USN, District Public Works Officer, Thirteen Naval District on behalf of the Chief, Bureau of Yards and Docks, awarded a contract in the amount of $21,645,000 for construction of the first increment of the project on 24 December 1959. Award was made to a joint venture of the following firms: Manson Construction and Engineering Company, Seattle, Washington; J.A. Jones Construction Company, Charlotte, N.C.; Perini Corporation, Framingham, Mass.; Osberg Construction Co., Seattle, Washington. Captain P.M. Boothe, CEC USN, Public Works Officer of Puget Sound Naval Shipyard is the Resident Officer in Charge of Construction. The contract provides for completion within 1,065 calendar days after award, with a liquidated damage penalty of $5500 per calendar day for delay of completion. The caisson, which is not yet under construction (1960), is estimated to cost an additional $1,000,000.

TABLE 2.—SUMMARY OF MATERIAL QUANTITIES (ESTIMATED) FOR DRY DOCK NO. 6

Material	Unit	Quantity
Dredging		
Total	CY	550,000
Fill & Backfill		
Initial Fill	CY	451,000
Berm Fill	CY	515,000
Cofferdam Fill	CY	90,500
Backfill : CY 352,000		
Spillover : CY 153,000		
Excavation: CY 100,000		
From C'Dam : CY 45,250	CY	53,750
Total	CY	1,110,250
Sheet Pile		
Berm	Tons	3,050
Cofferdam	Tons	2,390
Total	Tons	5,440
Concrete		
Dry Dock	CY	151,000
Pier 9	CY	900
Substation #83	CY	1,100
Floodlight Towers	CY	255
Total	CY	153,255
Prestressed Concrete Pile		
Total	LF	12,700
Reinforcing Steel		
Dry Dock	Tons	8,300
Pier 9	Tons	115
Substation #83	Tons	80
Floodlight Towers	Tons	15
Total	Tons	8,510
Misc. Iron & Struct. Steel		
Dry Dock	Tons	360
Rails & Rail Plates	Tons	320
Switches & Frogs	Tons	70
Service Building	Tons	35
Floodlight Towers	Tons	81
Total	Tons	866
Rip Rap		
Total	CY	8,400
Drainage Course		
Total	CY	92,000
Bituminous Concrete Pavement		
Total	SY	32,500
Concrete Masonry		
Total	SF	4,400

Throughout the 3 yr of planning and design, close coordination was necessary to resolve a multitude of technical and administrative problems and to meet scheduled dates. This project exemplifies the fine Navy teamwork of the sponsor bureau, the Bureau of Ships; the Bureau of Yards and Docks, which has the responsibility for design and construction; the Puget Sound Naval Shipyard, the using activity; the District Public Works Officer; and the Architect and Engineer.

Without the full cooperation of all the members of the team, a project of this magnitude could not be accomplished.

The Carrier Repair Site, when completed in 1961, will provide West Coast facilities to service the largest fleet units that are in service, under construction, on the drawing boards or contemplated.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3150

HYDRODYNAMIC WAVE PRESSURE ON BREAKWATERS

By M. A. Gouda[1]

SYNOPSIS

This paper presents a theoretical solution to the problem of determining the hydrodynamic wave pressure on breakwaters. The pressures derived are in close agreement with those obtained by other methods that are based on observations and experiments, such as those by Luiggi, Sainflou, Cagli, and Benedit.

INTRODUCTION

The problem of wave pressure on breakwaters is of much importance to port and harbor engineers. Previous investigators discussing this problem based their work mostly on observations and experiments. Results are given in the form of diagrams and empirical formulae such as that given by Luiggi, Sainflou, Cagli, Benedit, and others.

An attempt is made herein to obtain a solution for this problem based totally on the theory of hydrodynamics. This theoretical investigation is of great practical value as engineers in increasing numbers are drawing extensively

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Note.—Published essentially as printed here, in March, 1960, in the Journal of the Waterways and Harbors Division, as Proceedings Paper 2414. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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on the results of mathematical analysis as an aid in their design work. Moreover, results obtained for a specific wave problem by applying the mentioned diagrams and equations are of such different values that a theoretical investigation is needed.

SURFACE AND STANDING WAVES

Consider the oscillation of a horizontal sheet of water where the depth is very great. The motion, being such as could be produced from rest by natural forces, is irrotational and the velocity potential ϕ has to satisfy the differential equation[2]

$$\frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

with the condition $\partial \phi / \partial n = 0$ at a fixed barrier in the z-x plane, where $\frac{\partial}{\partial n}$ denotes differentiation with respect to definite distance from the barrier.

The pressure at any point (x, y, z) at time t is given by:

$$\frac{p}{\rho} = - g z - \frac{1}{2} q^2 + \dot{\phi} + f(t) \ldots\ldots\ldots\ldots\ldots \quad (2)$$

where p is pressure, ρ denotes density of liquid, q is velocity, and f(t) represents an arbitrary function of the time which may be supposed merged in $\dot{\phi}$.

Thus the pressure will be

$$\frac{p}{\rho} = - g z - \frac{1}{2} q^2 + \dot{\phi} \quad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

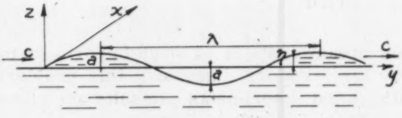

FIG. 1.—ELEMENTS OF A STANDING WAVE

Referring to Fig. 1 for a standing wave of amplitude a, wavelength λ, and velocity of propagation c, it is known that $c^2 = g/K$ where $K = 2\pi/\lambda$ and in deep water, the elevation of the free surface[3]

$$\eta = 2 a \cos (K y) \cos (K c t) \ldots\ldots\ldots\ldots\ldots \quad (4)$$

To arrive at a practical solution for the problem, the following assumptions have to be considered:

1. The barrier is assumed very long and thus the solution is independent of x.
2. The solution is periodic in the time t and the period being $\frac{\lambda}{c} = \frac{2\pi}{K c}$ also it is periodic in y, its period being $\lambda = 2\pi / K$.
3. At the barrier where $y = 0$ the horizontal velocity $v = \partial\phi/\partial y = 0$ also at the bottom of the sea where $z = -\infty$, the vertical velocity $w = -\frac{\partial \phi}{\partial z} = 0$.

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[2] "A Treatise on Hydro-Mechanics," by Arthur S. Ramsey, Hydrodynamics, Part II, pp. 267.

[3] "Theoretical Hydrodynamics," by L. M. Milin - Thomson, pp. 354.

4. At the free surface where $z = \eta$, the pressure $p = 0$. Hence by substituting in Eq. 3, $\phi - \frac{1}{2} q^2 = g\,z$.
5. The surface of waves is expected to be of the form

$$\eta = 2\,a \cos (K\,y) \cos (K\,c\,t) + \eta' \ldots\ldots\ldots\ldots \quad (5)$$

where η' is a function of y and t and $K = g/c^2$

SOLUTION OF THE PRESSURE DIFFERENTIAL EQUATION

The semi-inverse method is used to obtain the potential ϕ satisfying Eq. 3. The velocity v could be written in the form

$$v = V \sin (K\,y) \sin (K\,c\,t) = \frac{\partial \phi}{\partial y} \quad \ldots\ldots\ldots\ldots \quad (6)$$

and since

$$\frac{\partial^2 v}{\partial y^2} + \frac{\partial^2 v}{\partial z^2} = O \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

therefore

$$- K^2\,V \sin (K\,y) \sin (K\,c\,t) + \frac{d^2 V}{dz^2} \sin (K\,y) \sin (Kct) = O \ldots\ldots \quad (8)$$

or

$$\frac{d^2 V}{dz^2} - K^2\,V = O \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

thus

$$V = A_1\,e^{K\,z} + A_2\,e^{-K\,z} \ldots\ldots\ldots\ldots\ldots \quad (10)$$

Substituting from Eq. 10 into Eq. 6 yields

$$v = (A_1\,e^{Kz} + A_2\,e^{-Kz}) \sin (K\,y) \sin (K\,c\,t) \ldots\ldots\ldots\ldots \quad (11)$$

but since

$$\frac{\partial w}{\partial y} = - \frac{\partial^2 \phi}{\partial y\,\partial z} = - \frac{\partial}{\partial z} \frac{\partial}{\partial y} = \frac{\partial v}{\partial z}$$

$$= K\,(A_1\,e^{K\,z} - A_2\,e^{-K\,z}) \sin (K\,y) \sin (K\,c\,t) \ldots\ldots\ldots\ldots \quad (12)$$

and

$$\frac{\partial w}{\partial z} = - \frac{\partial^2 \phi}{\partial z^e} = \frac{\partial^2 \phi}{\partial y^2} = - \frac{\partial v}{\partial y}$$

$$= -K\,(A_1\,e^{K\,z} + A_2\,e^{-K\,z}) \cos (K\,y) \sin (K\,c\,t) \ldots\ldots\ldots\ldots \quad (13)$$

Thus from Eqs. 12 and 13 the vertical velocity w will be

$$w = -\left(A_1\, e^{K z} - A_2\, e^{-K z}\right) \cos (K y) \sin (K c t) \tag{14}$$

and since, from assumption 3, $w = O$ when $z = -\infty$, therefore $A_2 = O$. Hence the horizontal and vertical velocities v and w will be

$$v = A_1\, e^{Kz} \sin (K y) \sin (K c t) \tag{15a}$$

and

$$w = -A_1\, e^{Kz} \cos (K y) \sin (K c t) \tag{15b}$$

From previous expressions of the velocities and since $v = -\partial \phi / \partial y$ and $w = -\partial \phi / \partial z$ then

$$\phi = \frac{A_1}{K}\, e^{Kz} \cos (K y) \sin (K c t) + f(t) \tag{16}$$

or

$$\phi = A\, e^{K z} \cos (K y) \sin (K c t) + f(t) \tag{17}$$

where $A = A_1/K$, but since $q^2 = v^2 + w^2$, and by expressing values of v and w from Eq. 15, thus

$$q^2 = A^2\, K^2\, e^{2 K z} \sin^2 (K c t) \tag{18}$$

From expression of ϕ in Eq. 17,

$$\phi = \frac{\partial \phi}{\partial t} = A\, K\, c\, e^{K z} \cos (K y) \cos (K c t) + f'(t) \tag{19}$$

Substituting from Eqs. 18 and 19 into Eq. 3,

$$\frac{p}{\rho} = -g z - \frac{1}{2} A^2\, K^2\, e^{2 K z} \sin^2 (K c t)$$

$$+ A\, K\, c\, e^{Kz} \cos (K y) \cos (K c t) + f'(t) \tag{20}$$

At the free surface where z equals η and by expanding terms $e^{2 K z}$ and $e^{K z}$ in power series and neglecting high orders in Eqs. 18 and 19 thus

$$^{\bullet}\phi = A\, K\, c\, (1 + K \eta) \cos (K y) \cos (K c t) + f'(t) \tag{21}$$

and

$$q_o^2 = A^2\, K^2 \sin^2 (K c t) \tag{22}$$

and by substituting previous expressions in Eq. 3 for the free surface where $p = O$ and $z = \eta$, therefore

$$O = -\eta - \frac{1}{2}\, \frac{A^2\, K^2}{g} \sin^2 (K c t)$$

$$+ \frac{A\, K\, c}{g} (1 + K\eta) \cos (K y) \cos (K c t) + \frac{1}{g} f'(t) \tag{23}$$

so that the equation of the free wave surface will be

$$\eta = \frac{A\,K\,c}{g}\cos(K\,y)\cos(K\,c\,t)$$

$$+\frac{A\,K^2c}{g}\cos(K\,y)\cos(K\,c\,t)-\frac{1}{2}\,\frac{A^2K^2}{g}\sin^2(K\,c\,t)+\frac{1}{g}f'(t)\;\ldots\ldots \qquad (24)$$

Putting $2\,a = A\,K\,c/g = A/c$ (since $K = g/c^2$), therefore

$$\eta = 2\,a\cos(K\,y)\cos(K\,c\,t) + 2\,a\,K\cos(K\,y)\cos(K\,c\,t)$$

$$-\,2\,a^2\frac{g}{c^2}\sin^2(K\,c\,t)+\frac{1}{g}f'(t)\;\ldots\ldots \qquad (25)$$

In the last expression by equating the elevation of the wave's free surface η to the first term only as a first approximation,

$$\eta = 2a\cos(K\,y)\cos(K\,c\,t)\;\ldots\ldots \qquad (26)$$

By substituting from Eq. 26 into Eq. 25, to get a better second approximation,

$$\eta = 2\,a\cos(K\,y)\cos(K\,c\,t) + 4\,a^2\,K\cos(K\,y)\cos(K\,c\,t)\cos(K\,y)\cos(K\,c\,t)$$

$$-\,2\,a^2\frac{g}{c^2}\sin^2(K\,c\,t)+\frac{1}{g}f'(t) = 2\,a\cos(K\,y)\cos(K\,c\,t)+\frac{a^2g}{c^2}\Big[\cos(2\,K\,y)$$

$$+\,2\cos(2\,K\,c\,t)+\cos(2\,K\,y)\cos(2\,K\,c\,t)\Big]+\frac{1}{g}f'(t) = 2\,a\cos(K\,y)\cos(K\,c\,t)$$

$$+\frac{a^2g}{c^2}\Big[\cos(2\,K\,y)+\cos(2\,K\,y)\cos(2\,K\,c\,t)\Big]\;\ldots\ldots \qquad (27)$$

where

$$\frac{a^2g}{c^2}\,2\cos(2\,K\,c\,t)+\frac{1}{g}f'(t) = 0\;\ldots\ldots \qquad (28)$$

that is,

$$\frac{1}{g}f'(t) = -2\,a^2\,K\cos(2\,K\,c\,t)\;\ldots\ldots \qquad (29)$$

THE PRESSURE AND FREE SURFACE EQUATIONS

By substituting Eq. 29 into Eq. 20, the pressure at any point (z, y) at or away from the breakwater and at any instant t is:

$$\frac{p}{\rho\,g} = -z - 2\,a^2\,K\,e^{2\,K\,z}\sin^2(K\,c\,t) + 2\,a\,e^{K\,z}\cos(K\,y)\cos(K\,c\,t)$$

$$-\,2\,a^2K\cos(2\,K\,c\,t)$$

Putting $P = \frac{p}{\rho g}$ leads to

$$P = -z + 2a\,[e^{Kz}\cos(Ky)\cos(Kct) - aKe^{2Kz}\sin^2(Kct) - aK\cos(2Kct)] \qquad (30)$$

From Eq. 27 the free surface

$$\eta = 2a\cos(Ky)\cos(Kct) + 2a^2K\cos(2Ky)\cos^2(Kct)$$
$$= 2a\,[\cos(Ky)\cos(Kct) + aK\cos(2Ky)\cos^2(Kct)] \qquad (31)$$

From previous approximations until derivation of the final pressure and surface Eqs. 30 and 31, terms whose amplitude is of order (aK) were retained and higher powers were neglected.

This approximation is then satisfactory if these terms can be neglected numerically.

If $a^2K^2 \leq 0.09$ therefore $aK \leq 0.3$ and since $K = 2\pi/\lambda$ therefore $\frac{2a\pi}{\lambda} \leq 0.3$ or $\frac{2a}{\lambda} \leq 0.3/\pi$ and hence $\frac{\lambda}{2a} \geq \frac{\pi}{0.3} = 10.46$

This means that Eqs. 30 and 31 are applicable for all wave cases where

$$\frac{\text{length of wave}}{\text{height of wave}} = \frac{\lambda}{2a} \geq 10$$

Such limitation is within all practical values.

PRESSURE DISTRIBUTION ON BREAKWATERS (VERTICAL WALL)

The pressure on the breakwater can be obtained by substituting $y = 0$ in Eq. 30 for any time t.

$$P = -z + 2a\,[e^{Kz}\cos(Kct) - aKe^{2Kz}\sin^2(Kct) - aK\cos(2Kct)] \qquad (32)$$

The first term in Eq. 32 represents the hydrostatic pressure, while the second terms represent the hydrodynamic pressure. Thus, hydrodynamic-pressure equations will be:

$$P = 2a\,[e^{Kz}\cos(Kct) - aKe^{2Kz}\sin^2(Kct) - aK\cos(2Kct)] \qquad (33)$$

where $-\infty < z \leq 0$ and

$$P = -z + 2a\,[e^{Kz}\cos(Kct) - aKe^{2Kz}\sin^2(Kct) - aK\cos(aKct)] \qquad (34)$$

where $0 \leq 2 \leq \eta$ and

$$\eta = 2a\cos(Kct) + aK\cos Kct \qquad (35)$$

It should be noticed that the pressure distribution obtained by Eq. 33 is of great value when $t = 0$. This may be examined easily from the value of cos (K c t) which varies between a minimum of zero for $t = \frac{\pi}{2\,K\,c}$ and a maximum of unity for $t = 0$. Thus, at the breakwater for $t = 0$, the hydrodynamic pressure:

$$P = 2\,a\,(e^{Kz} - a\,K) \qquad (36)$$

where $-\infty < z \leq 0$, and

$$P = -z + 2\,a\,(e^{Kz} - a\,K) \qquad (37)$$

where $0 \leq z \leq \eta$ and the free surface for $t = 0$

$$\eta = 2\,a\,(1 + a\,K) \qquad (38)$$

In Fig. 2, the distribution of hydrodynamic wave pressure on a breakwater wall is given for a wave 5.0 m high and 75.0 m long at different times t.

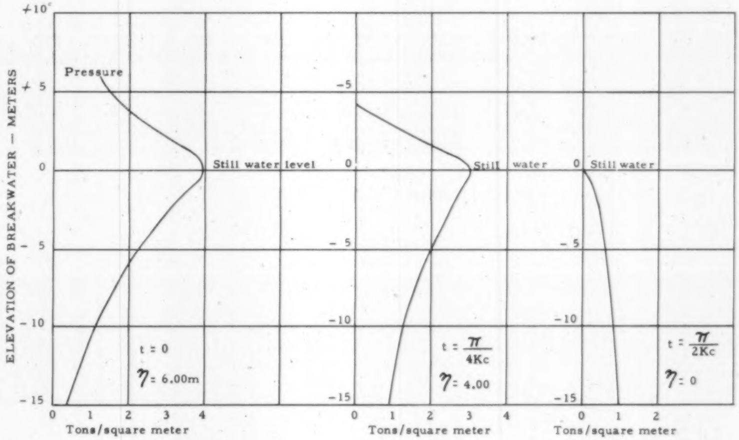

FIG. 2.—HYDRODYNAMIC PRESSURE ON BREAKWATER WALL (FOR A WAVE 5.00 m HIGH AND 75.00 m LONG AT DIFFERENT TIMES t)

PRACTICAL CURVES

Eqs. 36 and 37 give a picture of the distribution of the hydrodynamic pressure at the breakwater when the height of the free surface is limited by Eq. 38.

It may be seen from Eqs. 36 and 37 that the maximum pressure occurs at the mean sea water level, that is, at $z = 0$, and hence:

$$P_{max} = 2\,a\,(1 - a\,K)$$

and since $K = 2\pi/\lambda$

$$P_{max} = 2\,a\left(1 - \frac{2\,a\,\pi}{\lambda}\right) \qquad (39)$$

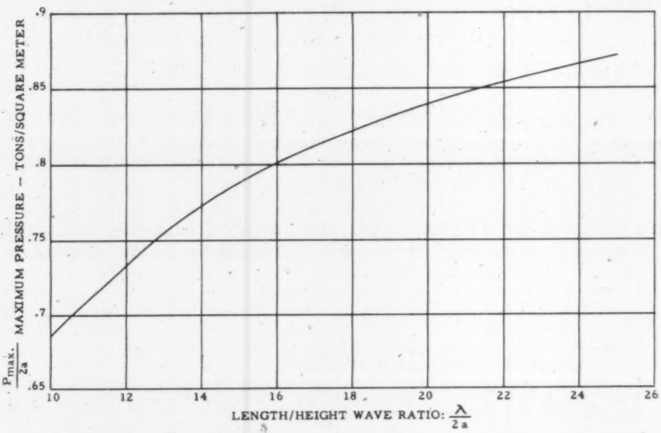

FIG. 3.—MAXIMUM HYDRODYNAMIC PRESSURE AT STILL WATER

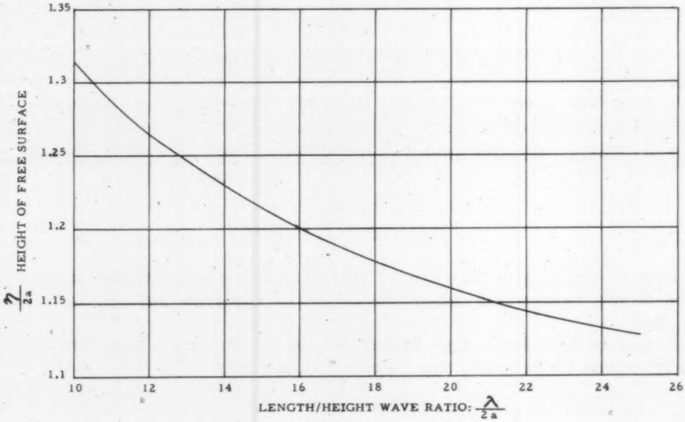

FIG. 4.—HEIGHT OF FREE SURFACE AT BREAKWATER

and

$$\eta = 2\,a\left(1 + \frac{2\,a\,\pi}{\lambda}\right) \ldots\ldots\ldots\ldots (40)$$

From Eq. 39 and 40, curves may be drawn relating the length/height wave ratio ($\lambda/2a$) to the maximum hydrodynamic pressure at the mean sea water level or to the maximum height of the free surface of the wave at the breakwater as shown in Figs. 3 and 4.

It may be noticed from Eqs. 36 and 37 that the hydrodynamic pressure P will be equal to zero when

$$e^{K\,z} = a\,K$$

or

$$z = \frac{1}{K} \log (a\,K) = \frac{\lambda}{2\,\pi} \log \frac{2\,a\,\pi}{\lambda} \ldots\ldots\ldots\ldots (41)$$

From Eq. 41, curves may be drawn to determine the depth of water at which the hydrodynamic pressure is equal to zero as shown in Fig. 5.

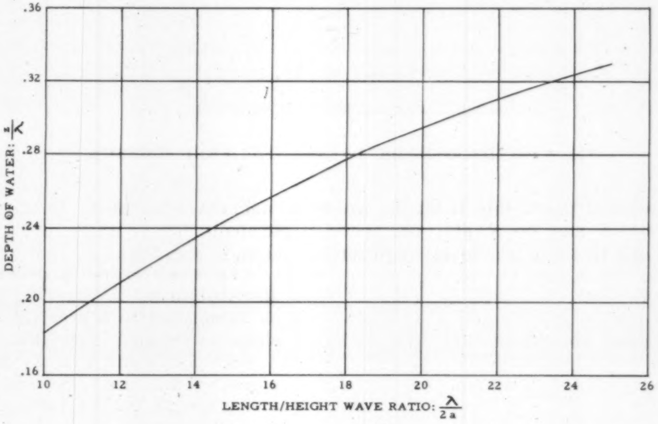

FIG. 5.—DEPTH OF WATER AT ZERO HYDRODYNAMIC WAVE PRESSURE

By substituting the height of the wave at the free surface from Eq. 40 into Eqs. 36 and 37, the value of the pressure will be

$$P_{\eta} = 2\,a \left[-\,\beta + \left(e^{4\,\pi\,a\,\beta/\lambda} - \frac{2\,a\,\pi}{\lambda} \right) \right] \ldots\ldots\ldots\ldots (42)$$

where $\beta = \left(1 + \frac{2\,a\pi}{\lambda}\right)$.

Eq. 42 may be used to plot curves relating the length/height wave ratio to the pressure at the free surface as shown in Fig. 6.

SIMPLE AND PRACTICAL WAVE PRESSURE DISTRIBUTION

The pressure distribution at the breakwater obtained by Eqs. 36 and 37 shows that the maximum pressure of the wave occurs at the mean sea water level in agreement with the results of Luiggi or Cagli.

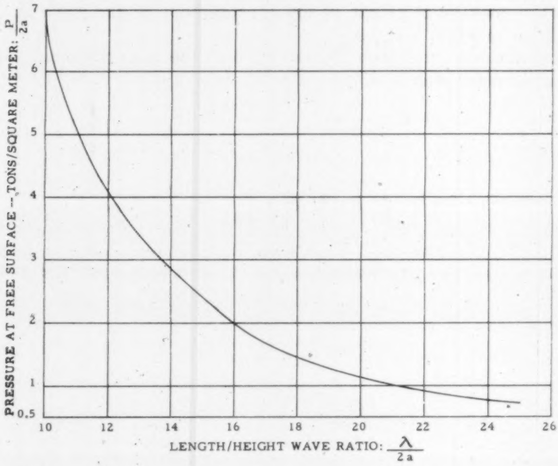

FIG. 6.—HYDRODYNAMIC PRESSURE AT FREE SURFACE

Above and below this level the pressure decreases in the form of some curve which may be roughly composed of parabolic (above water level) and hyperbolic (below water level) segments as shown in Fig. 7(a).

The exact shape of pressure distribution as in Fig. 7(a) may be simplified to a straight line shape as shown in Fig. 7(b).

Such a simplification is much more on the safe side and will give a simple and practical pressure distribution.

By the help of curves on Figs. 3, 4, 5, and 6, the straight-line diagram of pressure distribution may be determined at once for any wave. Considering the case of a wave 5 m high and 110 m long, the pressure distribution according to Luiggi, Cagli, and the writer's exact and simple straight line distributions are shown on Fig. 8.

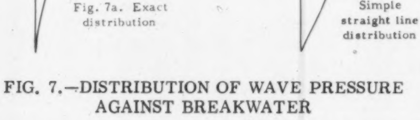

Fig. 7a. Exact distribution

Fig. 7b. Simple straight line distribution

FIG. 7.—DISTRIBUTION OF WAVE PRESSURE AGAINST BREAKWATER

DETERMINATION OF THE RESULTANT OF WAVE PRESSURE

In the design of breakwaters, it is very important to determine the amount and position of the resultant of wave pressure. Thus, by integrating in Eqs. 36 and 37 with respect to z from $z = -D$ to $z = \eta$ (where D = depth of breakwater), the value of the resultant wave pressure may be evaluated.

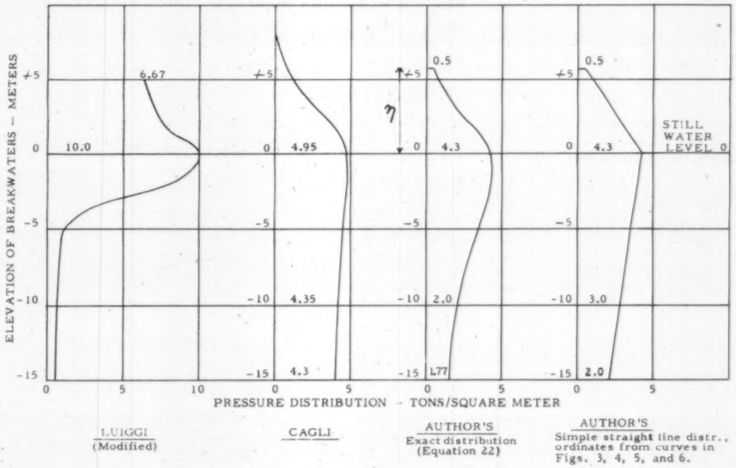

FIG. 8.—HYDRODYNAMIC PRESSURE DISTRIBUTION (FOR A WAVE 5.0 m HIGH AND 110 m LONG)

By adopting the straight-line pressure distribution and from Eqs. 39, 40, and 41, the value of the resultant wave pressure will be

$$Q = 2\,a\,(1 - a\,K)\left[a\,(1 + a\,K) + \frac{D}{2}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (43)$$

and since $K = 2\,a\,\pi/\lambda$

$$Q = 2\,a\left(1 - \frac{2\,a\,\pi}{\lambda}\right)\left[a\left(1 + \frac{2\,a\,\pi}{\lambda}\right) + \frac{D}{2}\right] \quad \ldots\ldots\ldots\ldots (44)$$

where $D >$ twice wave height.

In case of a deep breakwater where all wave pressure may be assumed acting

$$Q = 2\,a\left(1 - \frac{2\,a\,\pi}{\lambda}\right)\left[a\left(1 + \frac{2\,a\,\pi}{\lambda}\right) + \frac{\lambda}{2\pi}\log\frac{2\,a\,\pi}{\lambda}\right] \quad \ldots\ldots (45)$$

From Eq. 45, a curve may be drawn relating the value of the resultant wave pressure to the length/height wave ratio, as shown in Fig. 9.

Using the example of a wave 5.0 m in height and 110.0 m in length,

$$\frac{\lambda}{2\ a} = \frac{110}{5} = 22$$

From the curve (Fig. 9),

$$\frac{Q}{(2\ a)^2} = 3.4$$

and

$$Q = 3.4 \times 25 = 85 \text{ tons}$$

For comparison, using the Luiggi curve, Q = 90 tons.

CONCLUSIONS

Eqs. 36 and 37 will give the exact shape of hydrodynamic pressure distribution on a breakwater wall.

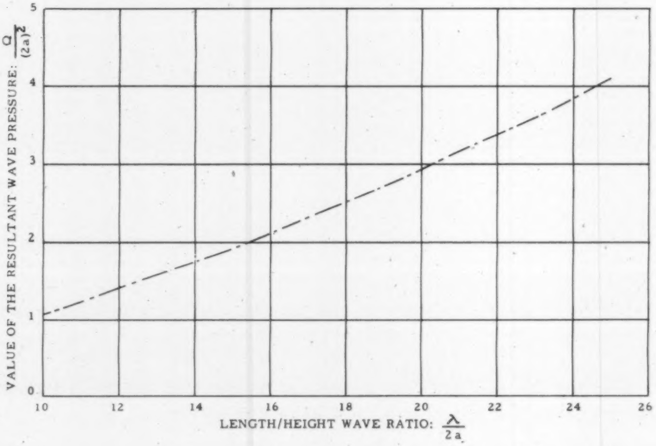

FIG. 9.—VALUE OF THE RESULTANT WAVE PRESSURE

The maximum intensity of pressure is at the still water level. (Values obtained by previous investigators Cagli, Sainflou, and Minikin do not differ much from those given by Eqs. 36 and 37.)

Above and below sea-water level, the intensity of pressure decreases in the form of a curve. (Values given by previous investigators below still water level are greater than those given by Eqs. 36 and 37.)

The maximum pressure distribution on a breakwater will be for time t = 0 (for the period = $2\ \pi / K\ C$).

The pressure distribution depends on the height and length of the wave.

Simple and practical pressure distribution may be obtained by following straight-line pressure diagrams using Figs. 3, 4, 5, and 6.

APPENDIX I.—ADDITIONAL REFERENCES

1. "Hydrodynamics" by Horace Lamb, Cambridge Univ. Press, 6th Edition, 1932.
2. "The Interruption of Surface Waves" by H. Y. Hammad.
3. "A Treatise on the Principles and Practice of Harbour Engineering" by Brysson Cunningham, Griffin, London, England, 3rd Edition, 1928.
4. "Winds Waves and Maritime Structures" by Robert C. Royal Minikin, Griffin, London, England, 1950.

APPENDIX II.—NOTATION

a = wave amplitude;

c = velocity of wave propagation;

D = depth of breakwater;

g = acceleration of gravity;

P = hydrodynamic wave pressure on a breakwater;

p = pressure;

Q = resultant wave pressure;

q = velocity;

t = time;

v = horizontal velocity;

w = vertical velocity;

β $= \left(1 + \frac{2 a \pi}{\lambda}\right)$;

η = elevation of free water surface;

λ = wave length;

ρ = density of liquid; and

ϕ = velocity potential:

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3152

APRON DESIGN FOR LIGHT AIRPLANES

By Kenneth K. Wilde,[1] A.M. ASCE

SYNOPSIS

This paper presents information for the design of parking aprons for single engine and light twin-engine aircraft. Standard dimensions are suggested for laying out tie-downs, and various geometric configurations are analyzed. The operational functions of the apron are then reviewed with respect to their needs and with respect to each other.

INTRODUCTION

In visiting numerous airports, the writer has become aware of the variations existing in the methods used for the tying down of light aircraft. The lack of any uniformity among these methods created an interest in finding if there was one best method. If such a design or method could be found, it would not only provide for easier operations, but benefits would also be derived from more efficient use of the apron or from reductions in the area constructed. It often appeared that only after an apron was constructed, was any thought given as to how it would be used. A typical result was that at one airport you might find a separation distance of 55 ft between rows of parked aircraft and 40 ft at the next. This paper, then, is partially directed toward suggesting separation distances and tie-down configurations for single-engine and light twin-engine airplanes.

Note.—Published essentially as printed here, in May, 1960, in the Journal of the Air Transport Division, as Proceedings Paper 2462. Positions and titles are those in effect when the paper or discussion was approved for publication in Transactions.

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DEFINITIONS

Before proceeding further, it is necessary to define precisely certain terms as they are used herein:

1. Tie-down anchor—The method or equipment used to hold physically an aircraft to the apron.
2. Tie-down point—The location on the aircraft to which the tie-down anchor is attached.
3. Tie-down spacing—The distances laterally and longitudinally between tie-down anchors for one aircraft.
4. Tie-down—The tie-down anchors necessary for the safe holding of one aircraft to the apron. (For the airplanes discussed herein, this is two wing anchors and one tail anchor.)
5. Tie-down configuration—The grouping of tie-downs for several aircraft to permit the efficient and safe operation of aircraft within a minimum of apron area.

TIE-DOWN ANCHORS

There are two general methods for the physical anchoring of the airplanes to the apron. For discussion, they shall be called the individual tie-down method and the cable tie-down method. The first of these uses an individual anchor for each tie-down point. In the second, a long cable anchored at periodic intervals is run the length of the apron, and the connecting cables from the wing tie-down points are attached to it. In both methods an individual tie-down anchor is used for the tail.

Both methods are in general use, and both work well. Advocates of the cable method claim that the cable slack acts as a shock absorber during gusty winds. It also has the advantage of being somewhat more flexible in that the separation distance between the wing tie-down anchors can be varied.

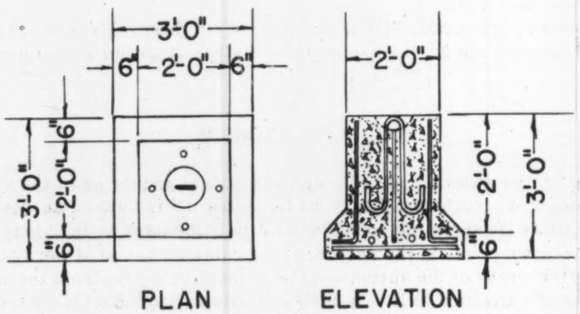
Regardless of the method used, a chain with a snap connector or a rope with a minimum tensil strength of about 700 lb should be used to make the connection between the anchor and tie-down point. Figs. 1 and 3 show designs for constructing anchors for both methods on an asphaltic concrete apron. Fig. 2 shows cable connector details and anchor spacing. For Portland cement concrete aprons, the bars used to form the connecting points are simply imbedded in the slab.

TIE-DOWN SPACING

When the method for anchoring the airplane has been selected, the separation distances between the wing anchors, and between the wing anchors and the tail anchor must be determined. This has been termed tie-down spacing. Tie-down spacing naturally is dependent on the distances between the tie-down points on the aircraft. Tables 1 and 2 list these distances for several popular types of aircraft.

Single-Engine Airplanes.—It would be desirable to select an average wing tie-down width separation which, when the anchor was connected to the tie-down point, would form an angle of about 45° to 60° with the pavement. This avoids a direct pull of the tie-down anchor, thus increasing the total force required to pull it out. The differences between high and low wing aircraft and the various models make this selection difficult.

An average of the wing-to-wing tie-down point distances in Table 1 yields a value of 17.2 ft. In order for an anchor rope to form a 60° angle with the apron for a high wing airplane, this distance would have to be increased by about 7 ft. The wing tie-down anchor separation thus would be 24 ft. Inasmuch as the single-engine fleet is comprised of approximately 85% high wing aircraft, and that inasmuch as for some of the low wing airplanes this distance is also adequate, 24 ft was adopted for the standard. Distances between 20 ft to 30 ft were observed in use at airports and found to be satisfactory.



END TIE-DOWN

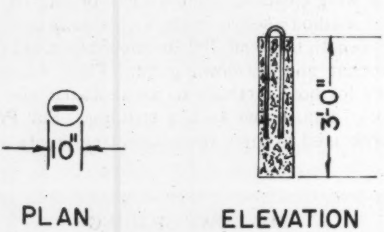
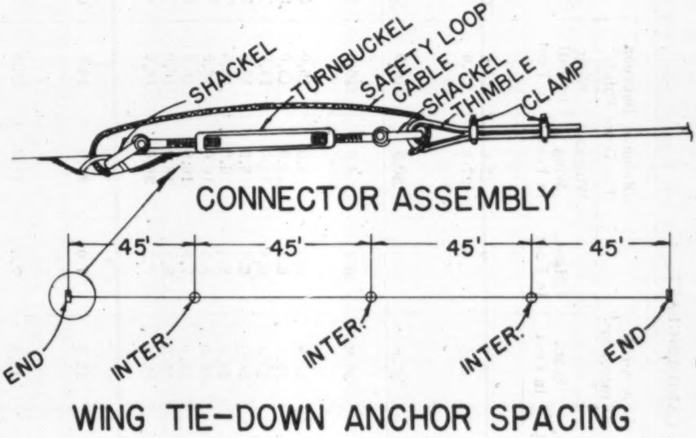


FIG. 1.—TIE-DOWN ANCHORS FOR USE WITH CABLES

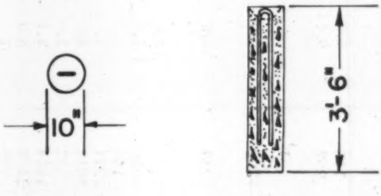
The wing-to-tail distance should also be sufficient to form an angle of 45° to 60° between the rope and ground, with the anchor being away from the aircraft. For aircraft with conventional gear configurations where the tie-down point is normally the tail wheel, this angle cannot be obtained. The average for the longitudinal distance between the wing tie-down points and the tail point for single-engine airplanes if 15.0 ft. To this should be added 2 ft to avoid a direct pull on the rope, making a standard length of 17 ft. Once again, distances varying between 15 ft and 20 ft were observed at airports, all of which were satisfactory.


SHACKEL
TURNBUCKEL
SAFETY LOOP
CABLE
SHACKEL
THIMBLE
CLAMP
45'
45'
45'
45'
END
INTER.
INTER.
INTER.
END

CONNECTOR ASSEMBLY

WING TIE-DOWN ANCHOR SPACING

FIG. 2.–CABLE LAYOUT



ANCHOR

NOTE: THIS ANCHOR IS USED FOR BOTH WING AND TAIL POINTS WHEN INDIVIDUAL ANCHORS ARE USED, AND FOR THE TAIL POINT WHEN CABLES ARE USED.

FIG. 3.–TIE-DOWN ANCHORS FOR INDIVIDUAL POINTS

TABLE 1.—SINGLE ENGINE LAND AIRCRAFT

Manufacturer and Model	Number of Active Aircraft[a]	Percentage Active of United States Total	Aircraft Dimensions[b]			Distance Between Tie-Down Points[c]		Gear Configuration
			Length, in Feet	Span, in Feet	Height, in Feet	Wing to Wing in Feet	Wing to Tail in Feet	
Aeronca Manufacturing Co.								
11AC, 11BC, 11CC (Chief)	116	1.8	20.4	36.1	7.0	22.0	14.5	Conventional
15AC (Sedan)	262	0.4	25.3	37.5	7.0	27.0	15.0	Conventional
Beech Aircraft Corp.								
35, A,B,C,D,E,F,G,H, & J,K, (Bonanza)	4202	7.0	25.2	32.8	6.5	20.5	16.0	Tricycle
Bellanca Aircraft Corp.								
1413, 14132, 14133	317	0.5	21.3	34.2	6.2	8.0	16.0	Conventional
Cessna Aircraft Co.								
120	1116	1.8	21.0	32.8	6.3	18.0	15.0	Conventional
140, 140A	3054	5.0	21.0	32.8	6.3	18.0	15.0	Conventional
150	100	0.2	21.0	33.3	6.9	13.0	14.0	Tricycle
170, 170A, 170B	3237	5.4	25.0	36.0	6.6	16.0	17.5	Conventional
172	2614	4.3	25.0	36.0	8.5	16.0	16.5	Tricycle
175	597	1.0	25.0	36.0	8.5	16.0	16.5	Tricycle
180, 180A, 180B	1874	3.1	26.0	36.0	7.5	16.0	16.5	Conventional
182, 182A, 182B, 182C	2109	3.5	26.0	36.0	8.5	16.0	16.0	Tricycle
190, 195, 195A, 195B	784	1.3	27.1	36.2	7.2	30.0	18.0	Conventional
Champion Aircraft Corp.								
7AC, 7BCM, 7CCM, 7DC, & 7EC, 7FC, 7GC (Aeronca)	3816	6.3	21.5	35.2	7.0	21.0	14.5	Conventional
Convair Stinson (Universal Aircraft Industries)								
108, 1081, 1082, 1083	2754	4.5	25.2	33.9	7.5	18.0	16.0	Conventional

Forney Aircraft Co.								
E, Fl, G, 415B, 415C, 415D, & 415CD, 415F, (Ercoupe)	2513	4.2	20.8	30.0	5.9	20.0	12.0	Tricycle
Mooney Aircraft Inc.								
M20	173	0.3	23.2	35.0	8.3	9.5[d]	12.0	Tricycle
M20A	110	0.2	23.2	35.0	8.4	9.5[d]	12.0	Tricycle
Piper Aircraft Corp.								
J3C65, J3F65, J3L65, J4A, & J5A (Cub)	4218	7.0	22.4	35.2	6.7	21.0	15.5	Conventional
PA15, PA17, PA18, PA18105SPEC, & PA18125, PA18135, PA18150, & PA18A, PA18A135 (Super Cub)	2064	3.4	22.4	35.3	6.7	--	--	Conventional
PA14, PA16 (Cruiser, Clipper)	582	1.0	23.2	35.5	6.4	15.5	14.0	Conventional
PA20, PA20135, PA22, PA22160, & PA22135, PA22150 (Pacer, Tripacer)	5513	9.1	20.3	29.3	6.2	16.0	13.0	(Conventional (Tricycle
PA24, PA24250 (Commanche)	552	0.9	24.7	36.0	7.4	19.5	13.5	Tricycle
Ryan Navion (Tusco Corp.)								
Navion, A, B, D	1297	2.1	28.7	33.4	8.7	8.5[d]	19.0	Tricycle
Silvare Aircraft Co.								
8 (A to F) (Luscombe)	2553	4.2	20.0	35.0	6.3	20.5	13.0	Conventional
Taylorcraft, Inc.								
BC12D, BC1265, BC65, BL65, & DC065, BC12DI	2325	3.9	22.0	36.0	6.5	21.5	15.5	Conventional
Universal Aircraft Industries								
GC1A, GC1B (Swift)	690	1.0	20.9	29.3	5.9	10.0[d]	13.5	Conventional
Total & Average this Table	50542	83.6%	23.3	34.4	7.1	17.2	15.0	
United States Total	60490	100%						

[a] As of Jan. 1, 1958, source of information, Statistical Study of U.S. Civil Aircraft. [b] Dimensions may vary between models and years. [c] Dimensions are to nearest ½ ft. [d] Distance between main gear.

Twin-Engine Airplanes.—A quick review of Table 2 shows that there is little consistency in the wing-to-wing separation distances for the twin-engine airplanes. The average distance is 26.0 ft; however, this is rather meaningless as the range runs from 13.0 ft to 40.5 ft. For certain of these airplanes the single-engine distance of 24 ft would work very well. Since twin-engine airplanes comprise less than 10% of all the active airplanes considered for this study, it therefore appears unnecessary to increase the 24 ft distance for them. The fact that most twins are low winged aircraft is of benefit here. The standard distance of 24 ft, therefore, also would be useable for the light twin-engine airplanes. If there was a sufficient number of twin-engine airplanes at the airport, however, and separate anchors could be justified for them, then a separation of about 32 ft would be in order. Since this distance varies greatly for twin-engine airplanes, it is recommended that a figure be selected most appropriate to the airplanes based at the airport.

There is little problem in the wing-to-tail distance, as it is nearly the same as that for the single-engine airplanes, the average distance being 18.0 ft, only 3 ft greater. This means that for a standard length of 17 ft, nearly a 90° angle will be formed. Although this is not desirable, it is workable as less lifting force is generated by winds at the tail section. For a twin-engine tie-down design, a distance of 21 ft is recommended.

TIE-DOWN CONFIGURATIONS

Before the tie-down anchors can be located on an apron, not only is it necessary to know the required spacing for the anchors for one tie-down of an individual airplane, but it is also necessary to know how the anchors for a number of airplane tie-downs can best be integrated. The object in laying out these tie-downs is to park the maximum number of airplanes in a given area, while still making it easy and safe to taxi and park the airplane. Many configurations are in use. Some of the best of these have been selected so that a comparison could be made of the area required per airplane for a particular configuration.

For laying out the configurations, a standard sized aircraft with a 35 ft wing span and a 25 ft length was used for the designs. This includes nearly all single-engine aircraft and a few of the smaller twin-engine aircraft. With the clearances used, it is also possible to park some of the larger twins. In order to avoid greatly reducing the clearance between aircraft while parking the twins, the airport manager should select two of the smaller single-engine aircraft to be parked adjacent to them. Dimensions of the aircraft in common use are included in Tables 1 and 2.

For certain designs, a comparison was made of the area requirements for single and double lane taxying. Double lane taxying allows for one taxying airplane to pass another. Approximately 30% more area is required for the latter type operation. Double lane taxying is required only when based aircraft number about 200, and even then it should only be adopted if a study of the apron area proves it necessary.

In designing the configurations, the following clearances were used:

1. Between two parked airplanes - 10 ft.
2. Between parked and moving airplanes - 15 ft (single lane taxying).
3. Between parked and moving airplanes - 10 ft (double lane taxying).
4. Between moving airplanes - 10 ft.

TABLE 2.—TWIN-ENGINE LAND AIRCRAFT

Manufacturer and Model	Number of Active Aircraft[a]	Percentage Active of United States Total	Aircraft Dimensions[b]			Distance Between Tie-Down Points[c]		Gear Configuration
			Length, in Feet	Span, in Feet	Height, in Feet	Wing to Wing in Feet	Wing to Tail in Feet	
Aero Design & Engineering Co.								
520	108	1.5	36.0	44.0	14.0	--	--	Tricycle
560, 560A,	131	1.9	35.4	44.1	14.75	13.0[d]	19.0	Tricycle
560E	32	0.5	35.4	49.0	14.4	--	--	Tricycle
680, 680E	202	2.9	35.4	44.0	14.5	13.0[d]	19.0	Tricycle
Beech Aircraft Corp.								
AT11 (Super 18) 18A, 18D, 18S	109	1.5	34.2	47.7	--	40.5	19.0	Conventional
B18S, D, D, E, (Twin Bonanza)	933	13.2	34.0	47.6	9.2	40.5	20.0	Conventional
50, B50, C50, D50, E50, & F50, G50 (Travel Air)	453	6.4	31.5	45.3	11.3	33.0	18.5	Tricycle
95	168	2.4	25.3	37.8	9.5	20.5	16.0	Tricycle
Cessna Aircraft Co.								
310, 310A, 310B, 310C	629	8.9	27.1	36.1	10.4	26.0	16.0	Tricycle
Piper Aircraft Corp.								
PA23, PA23160 (Apache)	1190	16.8	27.1	37.0	9.5	21.5	17.0	Tricycle
Temco Aircraft Corp.								
D16, D16A (Twin Navion)	74	1.0	27.2	34.0	10.3	--	--	Tricycle
Total & Average this Table	4029	57.0%	31.4	42.0	11.8	26.0	18.0	
United States Total	7073	100%						

[a] As of Jan. 1, 1959, source of information, Statistical Study of U.S. Civil Aircraft. [b] Dimensions may vary between models and years. [c] Dimensions are to nearest ½ ft. [d] Distance between main gear.

It was felt that there was some justification in reducing the clearance between parked and moving airplanes for double lane taxying, since the times that this will occur are limited, and the taxying speeds will be very slow. Based on the above clearances, an isle width of 65 ft is required for single lanes and 100 ft for double lanes.

Adoption for a configuration will depend on the following conditions:

a. Number of airplanes.
b. Direction and velocity of winds.
c. Area to be developed.

Figs. 4 through 10 show several apron designs patterned after designs in actual use at airports. All of the designs are usable. For larger airports where area is scarce it may be necessary to park the airplanes closer together, thus requiring a design as shown in Figs. 5 or 7. When space is not at such a great premium, a design like Fig. 6 can be used, eliminating any moving of airplanes by hand. This same design would be good in locations where high winds dictate that all aircraft be faced into the prevailing direction. When the area available for development is narrow or oddly shaped, it may be necessary to adopt a design that will best fit it.

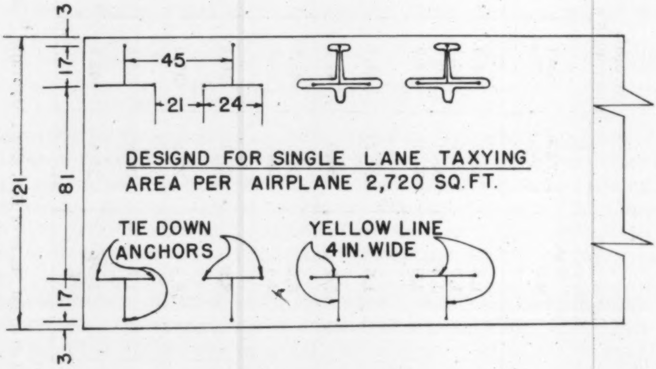
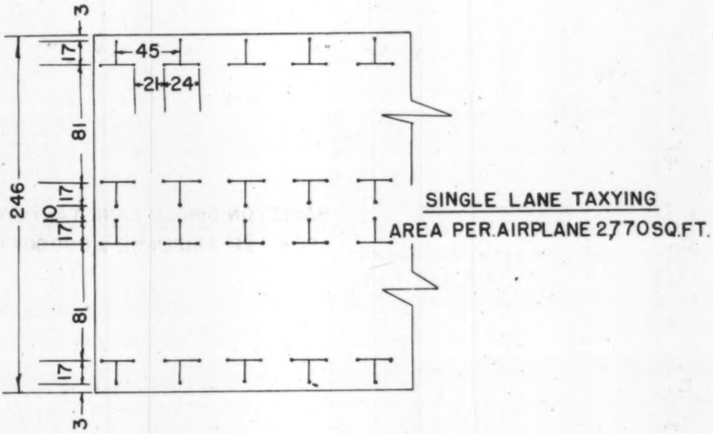

3
17
45
21
24
121
81
17
3
DESIGND FOR SINGLE LANE TAXYING
AREA PER AIRPLANE 2,720 SQ. FT.
TIE DOWN ANCHORS
YELLOW LINE 4 IN. WIDE

FIG. 4.—APRON DESIGN

It will be noted in Fig. 4 that a 4-in. wide yellow line has been shown connecting the tie-down anchors. Painting this line was found quite useful where it was in operation. It clearly shows the pilot where he should position his airplane and eliminates any mix-up in using the anchors by clearly defining each tie-down position.

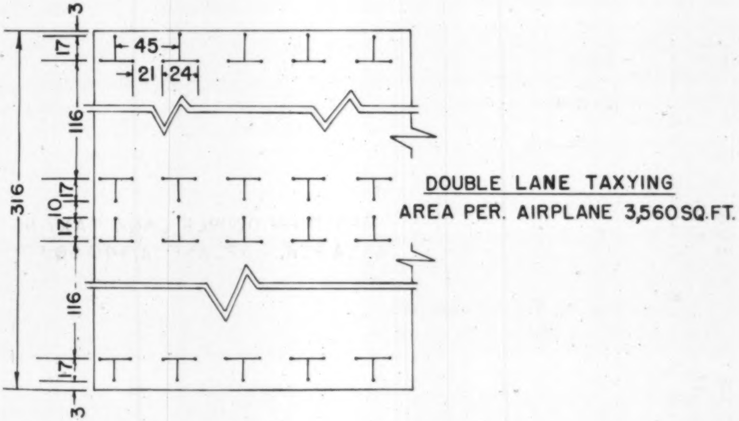
APRON SIZE AND LAYOUT

With the size and configuration of the airplane tie-downs established, the greatest remaining problem confronting the designer is, "How large should the apron be?" This, of course, is dependent on the activity at the airport. A logical approach to figuring this size is to analyze the individual needs of the parking apron.



3
17
45
21
24
81
246
17
10
17
81
17
3
SINGLE LANE TAXYING
AREA PER.AIRPLANE 2,770 SQ.FT.

FIG. 5.—APRON DESIGN



3
17
45
21
24
116
316
17
10
17
116
17
3
DOUBLE LANE TAXYING
AREA PER. AIRPLANE 3,560 SQ.FT.

FIG. 6.—APRON DESIGN

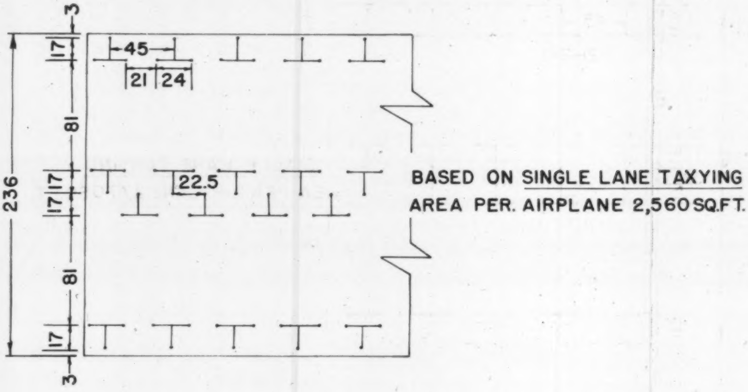

3
17
45
21
24
81
17
17
22.5
17
236
81
17
3
BASED ON SINGLE LANE TAXYING
AREA PER. AIRPLANE 2,560 SQ.FT.

FIG. 7.—APRON DESIGN

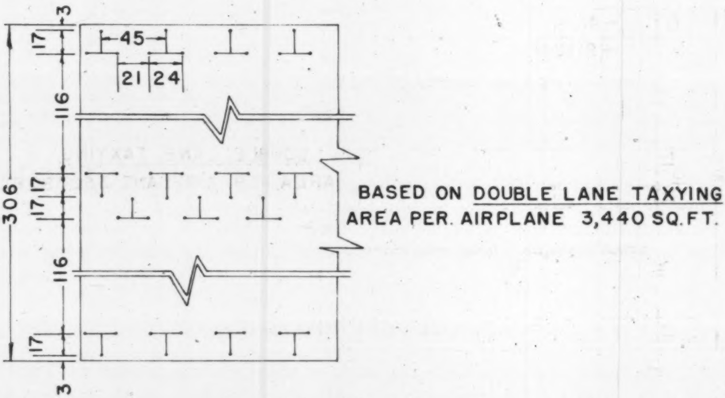

3
17
45
21
24
116
17
17
17
306
116
17
3
BASED ON DOUBLE LANE TAXYING
AREA PER. AIRPLANE 3,440 SQ. FT.

FIG. 8.—APRON DESIGN

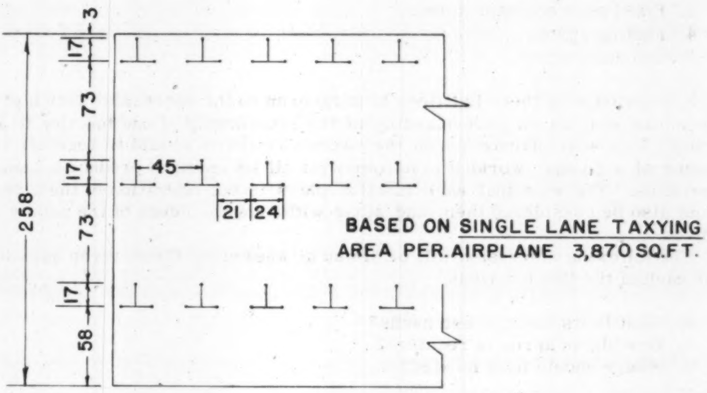

3
17
73
17
45
21
24
258
73
17
58
BASED ON SINGLE LANE TAXYING
AREA PER. AIRPLANE 3,870 SQ.FT.

FIG. 9.—APRON DESIGN

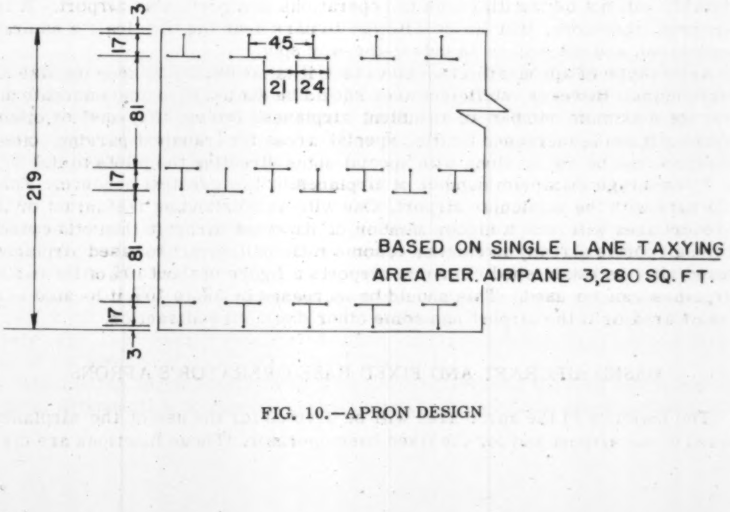

3
17
45
21
24
81
219
17
81
17
3
BASED ON SINGLE LANE TAXYING
AREA PER. AIRPANE 3,280 SQ. FT.

FIG. 10.—APRON DESIGN

The total apron area may be divided into the following five operational functions:

1. Based airplane apron.
2. Transient airplane apron.
3. Fixed base operator apron.
4. Fueling apron.
5. Unloading apron.

A knowledge of these functions is paramount to the successful design of an apron, as well as an understanding of the relationship of one function to another. In a well-planned apron the various functions should fit together like pieces of a puzzle, working in harmony for all its users to produce a smooth operation. The role that each function plays in the operation of the airport must also be considered then, and along with this the needs of the people involved.

The following discussion will be aimed at answering these three questions for each of the five functions:

a. What is its function and needs?
b. How big an apron is required?
c. Where should it be located?

Each of the five will be discussed individually.

TRANSIENT APRON

A certain portion of any apron must be devoted to transient airplanes. A transient pilot may be just stopping for fuel and food, for several hours, or over-night. Whatever his purpose, he should be extended the best service possible. This includes among others, a choice tie-down location on a paved apron with easy access to rest rooms, terminal building, and fuel. The transient pilot probably will not be familiar with the operations of a particular airport. It is pertinent, therefore, that he be allowed to park near the terminal, a natural destination, and not sent to an out-of-the-way spot.

As this type of apron will often be vacant, it is necessary to keep its size to a minimum. However, sufficient area should be set aside to accommodate an average maximum number of transient airplanes. During "Fly-ins" or other events attracting increased traffic, special areas for transient parking, often on grass, can be set up along with special signs directing the pilots to it.

The average maximum number of airplanes is the greatest concern. This will vary with the particular airport. One with an outstanding restaurant or in a resort area will have a higher number of itinerant aircraft than otherwise. If the airport is already in existence, some ratio of itinerant to based airplanes can readily be established. For new airports a figure of about 10% of the based airplanes can be used. This should be increased by 5% to 10% if located in a resort area or if the airport has some other distinctive attraction.

BASED AIRCRAFT AND FIXED BASE OPERATOR'S APRONS

The majority of the apron area will be devoted for the use of the airplanes based at the airport and for the fixed base operator. These functions are discussed together, for the apron areas are often used jointly. The fixed base operator's airplanes are normally counted and tied down along with those of private owners. He may even rent his tie-down space by the month based on actual need, rather than pay a flat rental for use of the apron.

For these two functions, the location need not receive as high a priority with respect to the terminal building and other facilities. The pilots know the airport, where they are going, and except for fuel, may not need other facilities. At larger airports, it may even be necessary to locate part of the based aircraft apron at a spot remote from the terminal area. It is necessary, however, to have adjacent automobile parking facilities, so that all such traffic can be kept clear of the operational areas.

In addition to his needs for storage of airplanes, the fixed base operator will require a certain amount of apron adjacent to his hangar if he offers repairs or other service. This will be a clear area, probably without tie-down anchors, used only for the intermittent parking of airplanes while servicing them and for entrance to his hangars. Anticipating the needs of the operator is difficult, as it depends on the business he generates. It is sometimes best to lease him land adjacent to the apron, and to allow him to construct his own apron. This shifts the burden of deciding how much apron to build from the airport owner, and also relieves him of the construction costs. The airport owner, however, should retain the right to direct the operator as to where he will build the apron. No matter who owns or uses an apron it must have continuity with the airport Master Plan.

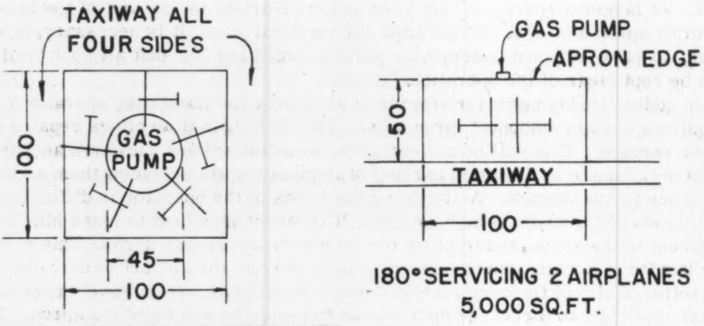
For planning the size of the combined aprons the number of based airplanes (including the fixed base operator's) should be anticipated for the next one to two years. A certain percentage of the owners of these airplanes will desire hangars, if these are available. In the most favorable of climates this percentage will be in the order of 10% to 20% of the based airplanes. It will naturally increase for more severe climates, such as ones with high winds or heavy snows. Another factor affecting this percentage is the value of the airplanes. For newer airplanes or twin-engine airplanes the cost of a hangar rental is not too great a proportion of the overall operating expense.

FUELING APRON

If a service station for refueling airplanes is to be used, then an apron area for this purpose must be planned. The choice of a service station or truck system operation is often one of personal preference and must be decided prior to the design of the apron system. Stations nearly always are used on airports of low and moderate activity.

A service station is most efficient if it is able to service airplanes on all sides or a 360° servicing area. This allows for a maximum number of airplanes to park near it with a minimum of moving of airplanes by hand, if any. With this design, up to five airplanes can be parked within range of the pumps, and require no hand moving. On airports of low activity, however, this design will take too great a percentage of the area, and also is not required. In this case, a pit adjacent to the apron and capable of fueling two airplanes is adequate. Fig. 11 shows the above two designs with the area required for the parking of the adopted standard 35 ft by 25 ft airplane.

The fueling apron should be located near the terminal building and transient apron. Attendants who may have other duties in the terminal can then watch the


TAXIWAY ALL
FOUR SIDES
GAS
PUMP
100
45
100
GAS PUMP
APRON EDGE
50
TAXIWAY
100

360° SERVICING 5 AIRPLANES
10,000 SQ.FT.

180° SERVICING 2 AIRPLANES
5,000 SQ.FT.

FIG. 11.—FUELING APRON

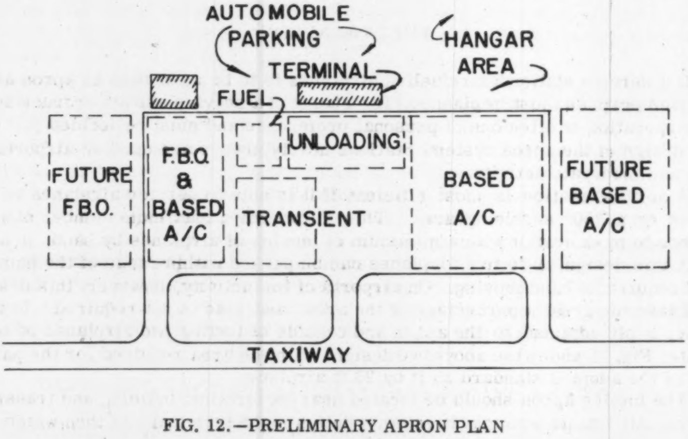

AUTOMOBILE
PARKING
TERMINAL
HANGAR
AREA
FUEL
UNLOADING
FUTURE
F.B.O.
F.B.O.
&
BASED
A/C
TRANSIENT
BASED
A/C
FUTURE
BASED
A/C
TAXIWAY

FIG. 12.—PRELIMINARY APRON PLAN

station and quickly give service when needed. With this location it is easy also for the attendant to park by hand any airplane left at the station for fueling and for tying down.

UNLOADING APRON

At larger general aviation airports, there is often a need for an unloading apron. This apron will be used by charter aircraft for discharging or enplaning passengers, for business aircraft, or for anyone who wants to stop for a few minutes to complete some business. Its size should be adequate to accommodate two average airplanes or one of the larger twin-engine type airplanes. Its location should be directly in front of the terminal building.

PLANNING AN APRON

The apron area usually is best located near the middle of the runway and on the side nearest the main road. Terrain or airport boundaries, however, may dictate that this practice be changed. When this is the case, all that can be done is to select the most level area affording the greatest expansion with a minimum of grading costs.

The size of the complete airport apron will be determined by the sum of the needs of the five functions previously considered. In designing this, the various functional aprons should first be interrelated. Fig. 12 shows one way in which this can be done. Each airport will require its own analysis. Approximations as to the overall size are sufficient to start with. After a basic plan has been established, taxiways can be added, tie-down configurations adopted, and exact dimensions given to the areas to accommodate the required aircraft.

Provision should always be made in the beginning for additional growth of the apron. It is usually unwise, therefore, to place buildings at the apron ends, as this is often the only direction for expansion. The tie-down configuration should also be chosen with expansion in mind. If not, it may be later necessary to waste area or to remove and to install new anchors.

CONCLUSIONS

Prior to beginning the design of an apron, the designer should first analyze the needs of the airport. This should encompass the type, size, and number of aircraft that it must support. From this information he will be able to select standards which will insure efficient operation. The only remaining act, then, is to tailor-make the apron to fit the airport.

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AERIAL PHOTOGRAPHY IN ARCTIC AND SUBARCTIC ENGINEERING

By Robert E. Frost,[1] M. ASCE

SYNOPSIS

The analytical airphoto procedures developed in temperate climates are equally applicable to engineering problems in cold regions. Included are: (1) consideration of the regional environmental aspects of the area; (2) detailed study of the minute characteristics and configurations composing the pattern; and (3) recognition and evaluation of the surface configurations that are the result, specifically, of permafrost and severe frost activity. The paper illustrates the foregoing and reviews the present "state of the art."

INTRODUCTION

The purpose of this paper is to discuss the use of aerial photographs as a means of getting information about surface materials and their condition which will assist in solving problems of a scientific, engineering, or military nature in arctic, subarctic, and polar areas. This discussion is limited to: (a) a short review of permafrost and severe frost activity as an arctic - subarctic problem; (b) a brief review of the airphoto method of obtaining information; (c) determination and evaluation of the regional environmental aspects responsible for permafrost and severe frost action and related problems; (d) identity and significance of minute photo pattern features; (e) direct permafrost and frost activity indicators; (f) a typical analysis; and (g) a few concluding statements.

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PERMAFROST, SEVERE FROST ACTIVITY AND THE PROBLEM

Permafrost is a condition of earth materials, including bedrocks, which exists at temperatures continuously below freezing in a generally solid state. The areal extent of permanently frozen soils is quite large. About one fifth of the dry-land surface of the earth is underlain by permanently frozen soils. The results of deep drilling programs have shown that the permafrost condition varies in thickness from a few feet in milder portions of northern lands to over 1,000 ft in favored locations, in extreme northern areas. On a regional basis, the presence of permafrost in an area is largely dependent on climate. On a local basis the type of permafrost occurring is largely a function of the natural and physical local environmental features. This includes topographic position, landform, slope, exposure, vegetative cover, surface drainage, and soil-parent materials. Evaluation of these features, whether accomplished by ground inspection or by the use of airphotos, is of paramount importance in planning any activity which will disturb the surface features in arctic and subarctic regions.

With regard to the effect of permafrost on engineering design, location, and construction it has been found that both good and bad soil situations occur in permafrost regions. The importance and severity of permafrost as a problem condition is largely a function of the ice-soil relationship in that portion of the soil/rock mantle having a temperature below freezing. For the most part the good construction areas contain well-drained granular materials which occur in elevated positions in comparison to the surrounding terrain. Coarse-grained soils (boulders, gravels, coarse sands) having little or no moisture content may exist in a loose, friable state even though the temperature may be below freezing (Figs. 1 and 2). In depressed or poorly drained positions, such soils may be saturated and may exist in a solid state because of the frozen interstitial water. In either event such soils can withstand thaw and disturbance without causing any appreciable difficulty.

The most adverse permafrost condition is that related to fine-textured soils containing a large percentage of ice in the form of crystals, lenses, wedges, sills, or massive accumulations of ice (Figs. 3 to 6). When such are situated low topographically or on extremely flat areas, the permafrost problem becomes extremely critical. Identification of this latter condition is important because thawing of this type of frozen soils is accompanied by severe volume change and loss of supporting power. Often the structures will experience severe distress almost immediately following construction (Figs. 7 and 8). Thaw is usually continuous under heated structures which have not been insulated properly from the ground surface. The destruction, or merely disturbance, of the protective natural insulation renders such frozen areas non-useable for structures and non-trafficable for most vehicles (Figs. 9 and 10). Recovery of permafrost of this type in disturbed areas is extremely slow, even after completely abandoning a disturbed site.

The permanently frozen soil mass is overlain by an "active layer" which freezes and thaws every year. The active layer is critical from the standpoint of movement of soils contained therein due to frost activity, gravity, solifluction, and other forms of earth-movement processes. In areas where a high percentage of silt occurs in the active zone, consideration of the active zone becomes more important than considerations of the permafrost zone for certain types of structures, particularly those structures which cannot withstand seasonal movement for successful performance. It is important that the properties of the active zone be determined so that disturbance of the often delicate

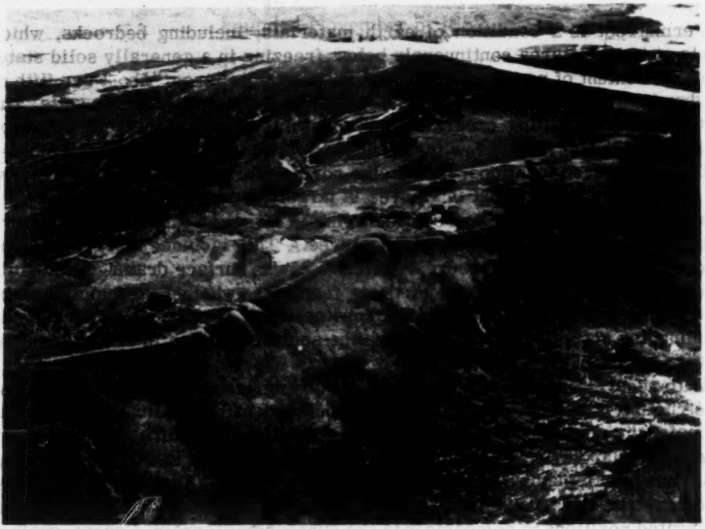

FIG. 1.—PORTIONS OF THIS GRANULAR TERRACE ARE WELL DRAINED AND UNFROZEN

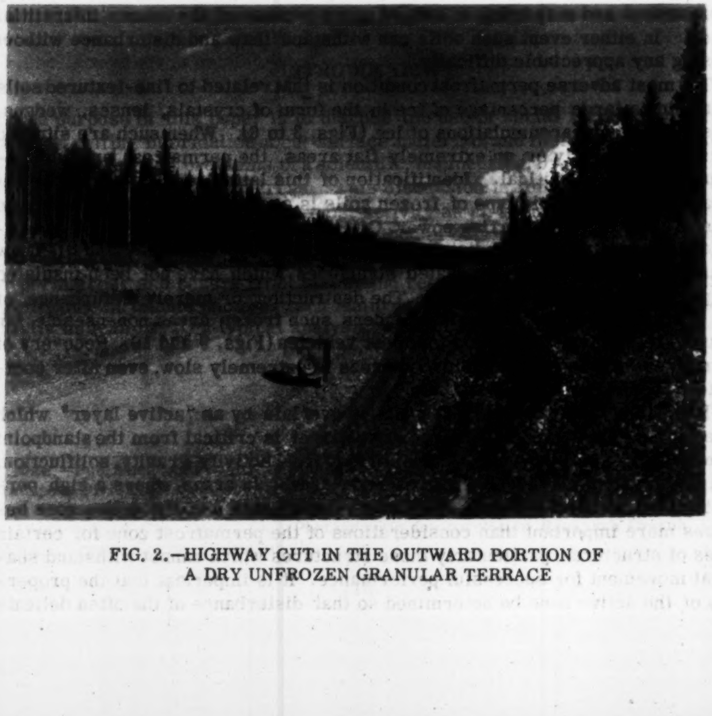

FIG. 2.—HIGHWAY CUT IN THE OUTWARD PORTION OF A DRY UNFROZEN GRANULAR TERRACE

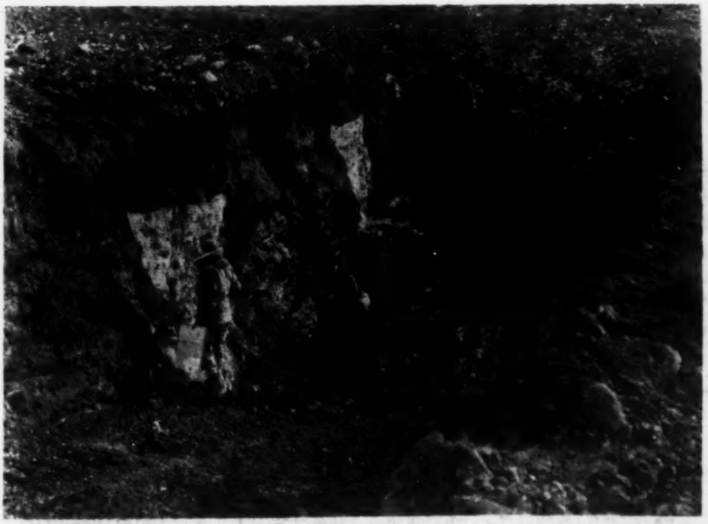

FIG. 3.—TEST PIT ACROSS A POLYGON EXPOSING ICE WEDGES WHICH OCCUR BENEATH POLYGON CHANNELS

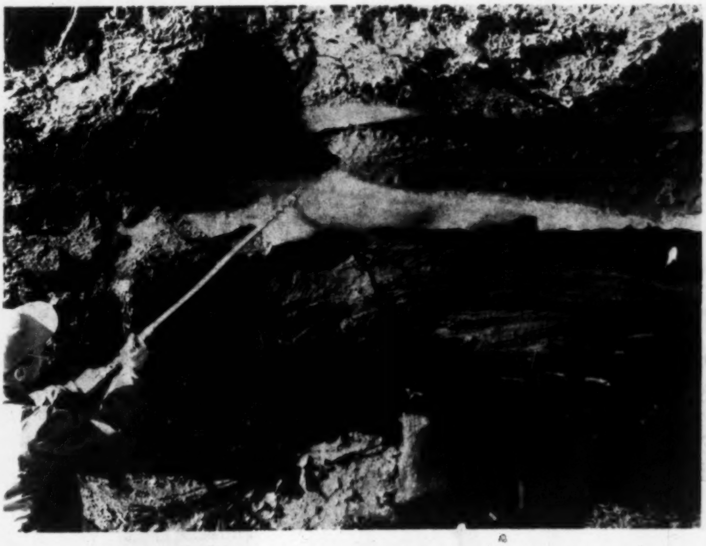

FIG. 4.—LEDGES OF ICE IN SILL FORM IN VALLEY FILL DEPOSITS

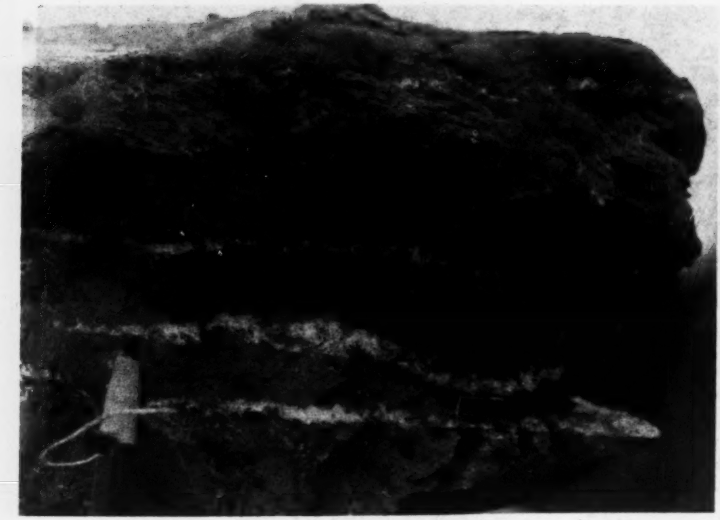

FIG. 5.—ICE CRYSTALS IN FINE GRAINED SOILS

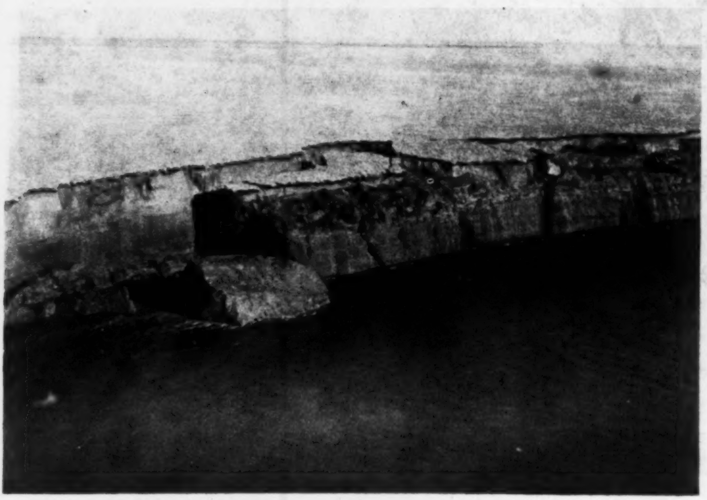

FIG. 6.—MASSIVE GROUND ICE ON FROZEN SAND AND SILT BEING UNDER CUT BY THAW ACTION OF THE WATER

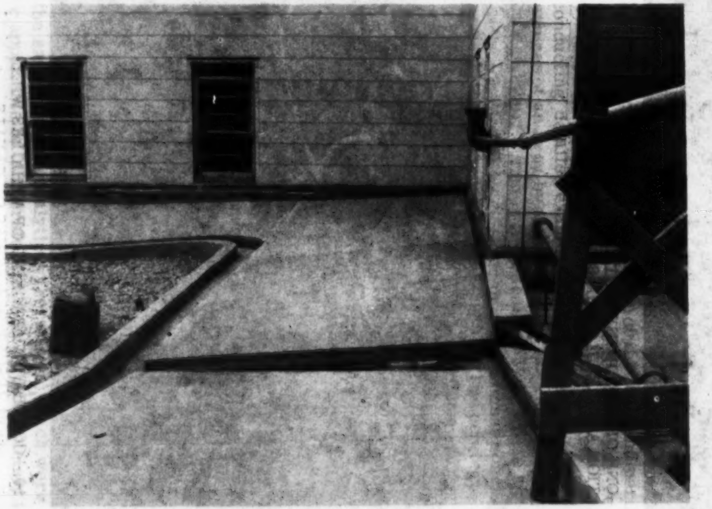

FIG. 7.—THE EXTENT OF DIFFERENTIAL MOVEMENT BETWEEN THE BUILDING AND ITS SURROUNDINGS INDICATES THE SENSITIVITY OF THE PERMAFROST THAW PROBLEM

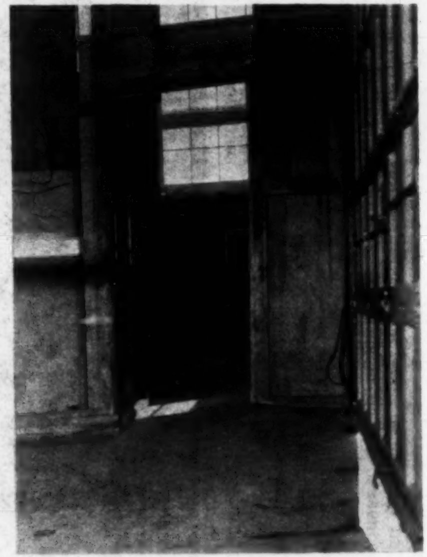

FIG. 8.—AN EXAMPLE OF VERY SEVERE SETTLEMENT IN A BUILDING RESULTING FROM THAW

FIG. 9.—ONE OR TWO PASSES WILL DESTROY THE INSULATION AND RESULT IN A SEVERE THAW

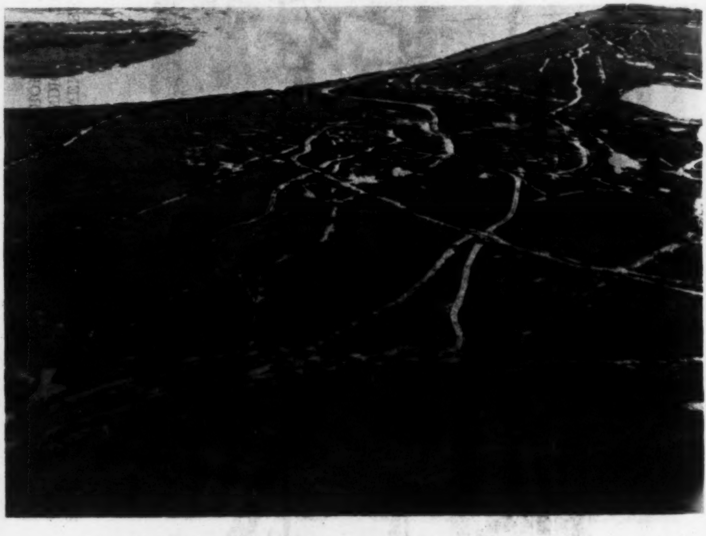

FIG. 10.—CONTINUED TRACKING OF VEHICLES WILL SOON RESULT IN A SERIES OF CANALS

thermal regime can be minimized. In an undisturbed state the active layer, together with its protective cover, plays an important part in preserving the permanently frozen layer by protecting it from the summer heat.

The engineering problems are contingent on the presence or absence of permafrost, which type exists, susceptibility of the active zone to severe frost activity, and where the structure is with respect to the permafrost-active zone profile. Structures confined to the active zone are subjected to seasonal freezing and thawing which will be accompanied by considerable movement if the soils are finely textured. This is particularly objectionable for such structures as highways, railroads, power lines, buildings, pipe lines, and utilidors. The period of most severe damage is during and shortly following the spring breakup. It is during this period that paved and unpaved road and airfield surfaces suffer the greatest distress (Fig. 11). If situated high topographically, such areas may recover rapidly once the frost leaves and the water drains away. If situated in depressed or flat topographic positions such soils may never recover.

Structures placed on frozen materials having low moisture content do not usually experience serious damage from the resulting thaw. Airfields, runways, roads, and buildings have been placed on frozen sands and gravels and have performed satisfactorily not only following construction but throughout the critical breakup seasons as well (Fig. 12). There are, however, often local areas containing much ice which lies within the bounds of dry frozen materials which should be avoided in engineering construction.

Identification, analysis, and evaluation of the soils and permafrost occurring in an area or deposit in the arctic, subarctic, and polar regions being considered for engineering use becomes a critical matter. Often time does not permit conducting detailed examination of the natural and physical environmental features of an area by field procedures. The far northern latitudes experience a very short summer field season. Many areas which offer good potential for the location of large structures are almost completely inaccessible to present day transportation means (1960). The strategic importance often creates a great urgency for obtaining information about remote areas which can be used in all problems concerned with earth-surface features. It is believed that the present "state of the art," or present state of development of airphoto analysis and interpretation, fulfills the majority of these requirements.

AIRPHOTO METHOD OF OBTAINING INFORMATION

The more reliable methods of obtaining information are based on research which was designed to coordinate aerial imagery with ground sampling in order that the many variables affecting information content in a picture could be defined and evaluated. Aerial images utilizing a broad range of the visible portion of the spectrum had to be tested for their faithfulness of representation of ground features within the wave length spread utilized by various films and filters and the system used. This research has proven fruitful from the standpoint of conventional optical photography. Interesting and significant trends are being established for sensor systems which seek information far beyond the visible portion of the spectrum.

The use and application of aerial photographs in arctic and subarctic areas follows the same general analytical procedures which were developed in and which are applicable to temperate climates. The completely analytical method

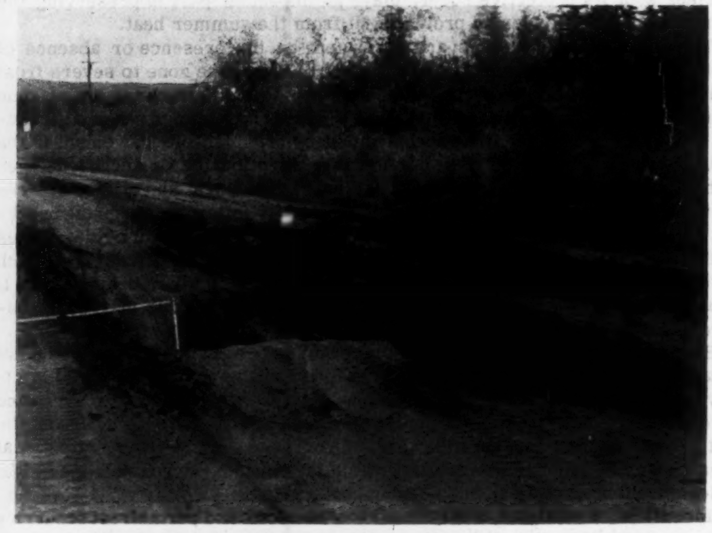

FIG. 11.—SEVERE ROAD FAILURE RESULTING FROM THAW OF FROST HEAVE IN SILTY SOILS

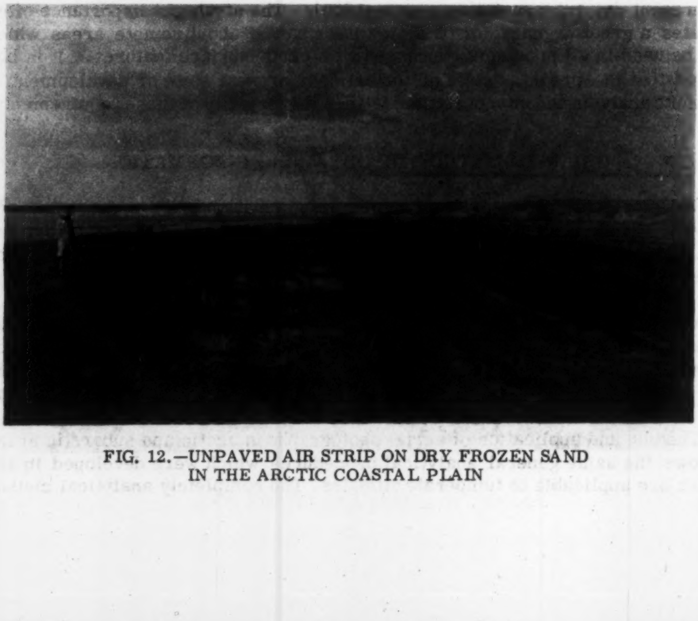

FIG. 12.—UNPAVED AIR STRIP ON DRY FROZEN SAND IN THE ARCTIC COASTAL PLAIN

of using airphotos for obtaining information about the soil mantle and its condition (frozen, frost acting, etc.) consists of applying logic and deductive reasoning to: (1) consideration of the regional environmental aspects of the area, (2) detailed study of the minute characteristics and configurations composing the pattern; and (3) recognition and evaluation of the surface configurations which are the result specifically of permafrost and/or severe frost activity.

REGIONAL ENVIRONMENTAL ASPECTS

Regional consideration includes geography, physiography, geology, and climate. The geographic aspects include physical and political location, size and shape of the area, boundary conditions, proximity to major land masses or water bodies, and major interest or cultural pursuit. Much of this, by necessity, must come from other sources.

The study of physiography is concerned with the relationships which exist between the major land units or land masses on a regional basis. The present landscape is an expression of all of the forces which have contributed to its development. These forces are related to deposition, erosion, orogenic movement, or any combination of the three. Consideration of the physiography of an area provides an important clue about the materials and their physical characteristics on a large scale. The arctic - subarctic landscape is altered to varying degrees by certain of the processes related to permafrost and severe frost activity. The most severe alteration occurs on land masses having a fine-grained soil and/or soil-rock mantle in which the confined forces of freeze-thaw and gravity result in severe surface movement. In such instances, the arctic landscape appears considerably rounded—slopes are soft and marks of movement occur everywhere. Coarse textured soil mantle and/or resistant rocks create the most rugged forms and slopes. In sloping areas the upper layer or top of the permafrost often acts as a good slipping zone or plane over which the materials in the active zone can move.

From the standpoint of geology, the analyst is interested in origin of deposits and the sequence of events responsible for development of the landscape. Each major parent material, whether developed in place and considered residual or transported by the action of wind, water, ice and/or gravity, has its own identifying pattern characteristics. Determination of the geology limits the type of materials expected to be found as well as degree and type of permafrost - frost activity in an area.

If meteorological data are not available the general climatic condition can be obtained from the airphotos but by inference only through the study of vegetation (its type and distribution) and by study of the erosional features. Knowing the general climatic condition under which the area has existed and under which the soils have formed from a given parent material gives an additional clue to the type of soil, the profile development, and, to a limited extent, permafrost and the susceptibility of seasonal frost activity. If the regional analysis has been successful, then the analyst knows something of location, size, and major area interest; has learned much about the physical expression of the major topographic forms; knows how the deposits were formed and what processes were active in shaping the area and what the major parent materials are, and; from the climatic indications, he has a good idea of presence or absence of permafrost and/or the severe seasonal frost potential. The regional study results in dividing an area into major patterns which are usually coincident with landform - parent material types. The landform, parent material types provide a basis for understanding what to expect with respect to problems related to location, design, construction, and, possibly, maintenance of large structures.

PHOTO PATTERN FEATURES

Each earth material is identified by the pattern features or elements, which it reflects. The common elements include landform, drainage, erosion, vegetation, tones, cultural features, and any unique or special features. It is to be stressed that detailed stereoscopic study of pattern elements results in determination of physical characteristics of materials and their state or condition. The presence of permafrost and/or severe frost activity does exert considerable influence on the appearance of some of the pattern elements of fine-grained soil and/or some rock types.

Analysis of the landform results in obtaining information about the composition of a deposit both areally and vertically. Landforms related to coarse-grained materials are little altered by the presence of permafrost while those which contain fine-grained soils may be altered considerably (Figs. 13 and 14). The surface characteristics will be altered considerably by severe frost activity if the active zone contains fine-grained soils and a source of moisture. Topographic position is important in determining the type and degree of permafrost - frost activity. The high or upland situations associated with coarse-grained materials offer no particular problem. The upland areas which contain fine-grained soils in depressed position can be expected to offer serious difficulties. The most severe topographic positions are related to the transition zones between high and low areas where the low areas are receiving fine-grained mantle from higher positions. Slope and exposure are items which must be studied in great detail since they greatly influence permafrost and thickness of the active layer. In general, the active zone is much thicker on south-facing slopes than on the north-facing slopes. This is of considerable importance from the standpoint of workability of engineering materials situated on sloping ground.

The drainage pattern reflects such things as topographic arrangement, general porosity, relative depth of soil mantle (if present), type and depth of rock (if present), and the areal extent and structure of either mantle or rock beneath the mantle. The regional drainage plan or pattern is little altered by the presence of permafrost. Locally, the drainage plan is altered considerably due to the presence of permafrost. The active zone and the frost-activity problem does not significantly affect the major drainage networks.

Erosional aspects provide an important clue to composition, condition, and uniformity of a deposit. In warm environments such characteristics as the gully arrangement, cross section, and profile are related to soil texture, soil structure, profile development, slope, and state of ground. In general, changes in any of the reflected characteristics means changes in materials or condition. Permafrost alters the erosional aspects of fine textured soils completely.

For the most part frozen soils erode by thaw which can be induced either naturally or through activities of man. A sudden increase in surface water in hilly areas may result in severe damage or possible destruction of established polygons (Figs. 15 and 16). The channels of the polygons become gullies and

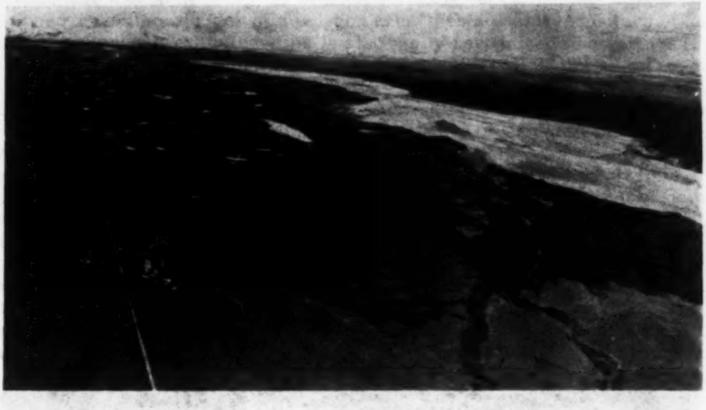

FIG. 13.—THE CHARACTERISTICS OF THE LANDFORM OF A WELL DRAINED GRANULAR TERRACE ARE LITTLE ALTERED BY THE PRESENCE OF THE PERMAFROST CONDITION

FIG. 14.—THE SURFACE CHARACTERISTICS OF PERMANENTLY FROZEN SILT COVERED TERRACES ARE ALTERED CONSIDERABLY BY THE PERMAFROST CONDITION

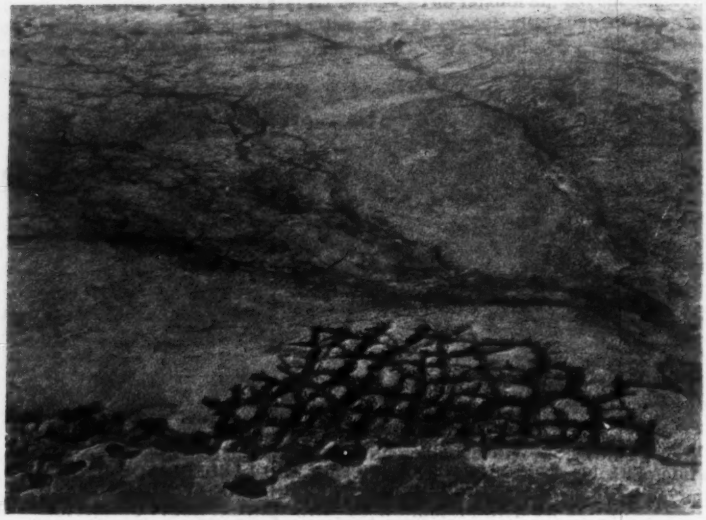

FIG. 15.—DESTRUCTION OF POLYGONS BY THAW

FIG. 16.—CONTINUED THAW OF POLYGONS RESULTS IN A SERIES OF SILT MOUNDS

destruction of the net by thaw and removal is often rapid. The ice in the channel thaws first causing a visible subsidence in the vegetation mat (Fig. 17). If thaw occurs on the surface of a polygon area then a drainage pattern of geometric regularity is exhibited—this is also easily recognized on airphotos (Fig. 18). Continued thaw and removal may result in leaving only isolated soil mounds as remnants.

Another common erosional feature in permafrost areas is the occurrence of "caving blocks" of frozen soils along the banks of streams and shores of large bodies of water (Figs. 19 and 20). The warm water of a stream, or lapping waves of the ocean, tends to induce waterline thaw which results in breaking off large blocks of frozen material. Thermokarst lakes provide another form of thermal upset in permafrost areas which results in a recognizable pattern on airphotos (Figs. 21 and 22). Thermokarst lakes are depressed basins in permafrost areas which are believed to have been formed by thawing of large masses of ground ice in an unstable permafrost regime. The lakes often are characterized by a scalloped or serrated shore line, and when occurring in timbered areas they are fringed by leaning trees (Fig. 23). In good quality airphotos it is possible to see overhanging mats of vegetation along the shore line of such lakes—another indication of thaw. One form of thermokarst pattern accompanies the sursurface thaw of polygons. The thawing action is influenced by ground slope, exposure, vegetative cover, soil texture, and surface water. If a polygon area has been upset by subsurface thaw, perhaps as a result of the destruction of the native cover, considerable subsidence occurs which results in a hummocky area having a pattern which is easily recognized on airphotos.

One of the most common types of gully systems in permafrost regions is that associated with "button drainage." Button gullies are usually found in association with polygon areas which are in the initial stages of thaw. Thawing usually starts at the intersection of polygon channels and small circular pools of water result. Usually flow is hardly perceptible. A chain of connected buttons forms the pattern (Fig. 23). The placer-type gully is another erosional feature common to permafrost areas and usually reflects undesirable permafrost conditions. Gullies of this type exist as broad softly rounded troughs which do not contain a definite channel. As long as such features are undisturbed, an upset by thaw will not result. Great quantities of water are stored in the vegetative mat, which retards the flow and prevents erosion.

Vegetation is an important pattern element as certain species are used as indicator species. The vegetation in an area provides a single expression into which are integrated all of the significant physical, chemical, and biological factors brought to bear upon it. Vegetative cover, which is of importance in arctic regions because of the protection afforded to the underlying permafrost, provides clues to type and degree of permafrost and conditions in the active zone. In timbered areas white spruce-paper birch forests occur on both frozen and unfrozen soils; black spruce-tamarack stands grow on frozen muskegs; aspen occurs on dry unfrozen, south-facing slopes; balsam-poplar stands are confined to sites adjacent to active streams having moist, sandy soils unfrozen to a depth of at least 10 ft or more; pure, dense, willow stands grow on bare river bars which are unfrozen to 10 ft or more; and pure alder brush occur on wet peaty soils frozen at a depth of 30 in. In tundra areas the cover includes niggerheads, mosses, sedges, grasses, lichens, berries, varieties of tea, dwarf birch, dwarf willow, low brush, and cotton sedges. Trees and large bushes are not found in tundra areas. Alders grow on slopes of hills and in major gullies, chiefly where protection from severe winds is afforded. Some willows,

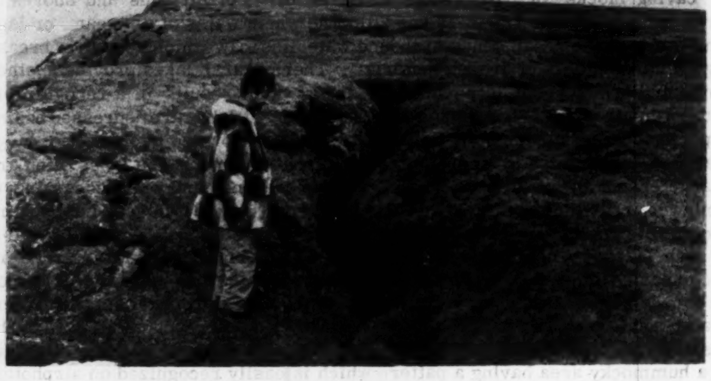

FIG. 17.—AS THE ICE IN A POLYGON CHANNEL THAWS THE OVERHANGING MAT OF VEGETATION IS LEFT UNSUPPORTED AND CRACKS BEGIN TO DEVELOP

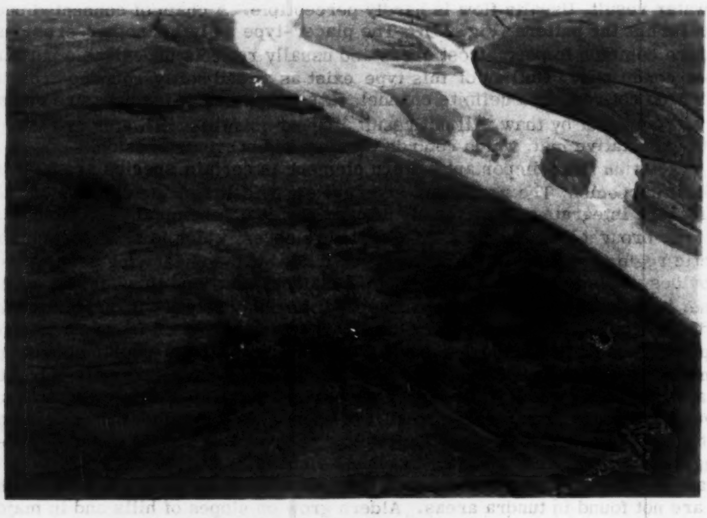

FIG. 18.—FREQUENTLY GULLIES EXHIBITING A GEOMETRIC DESIGN OCCUR IN POLYGON AREAS

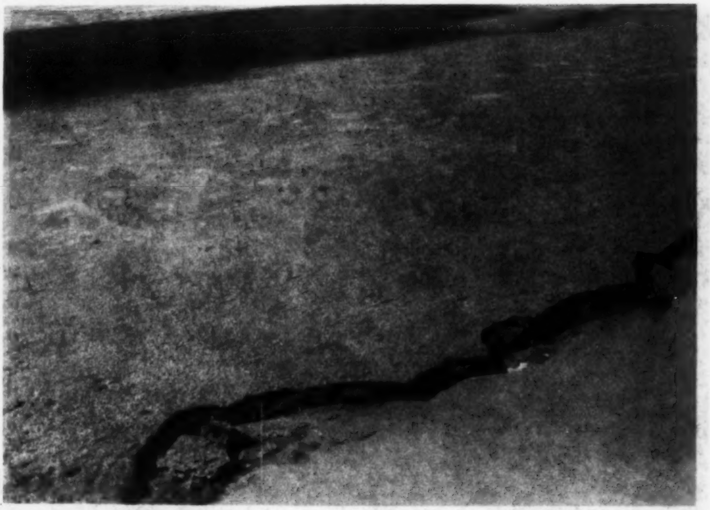

FIG. 19.—UNDERCUTTING OF STREAM BANKS BY THAW LEAVES LARGE BLOCKS OF FROZEN SOIL UNSUPPORTED

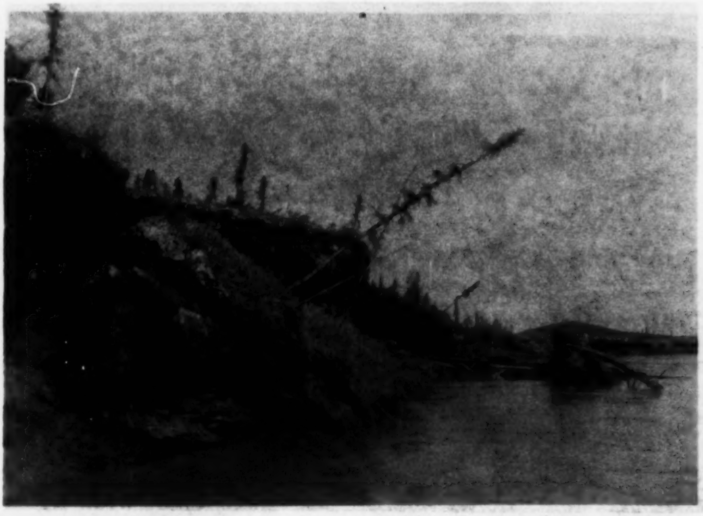

FIG. 20.—LARGE BLOCKS OF FROZEN SOIL WHICH HAVE BROKEN OFF FROM THE STREAM BANK

FIG. 21.—TWO THERMOKARST LAKES WHICH HAVE RESULTED FROM THAW OF MASSES OF GROUND ICE

FIG. 22.—SMALL THERMOKARST LAKES

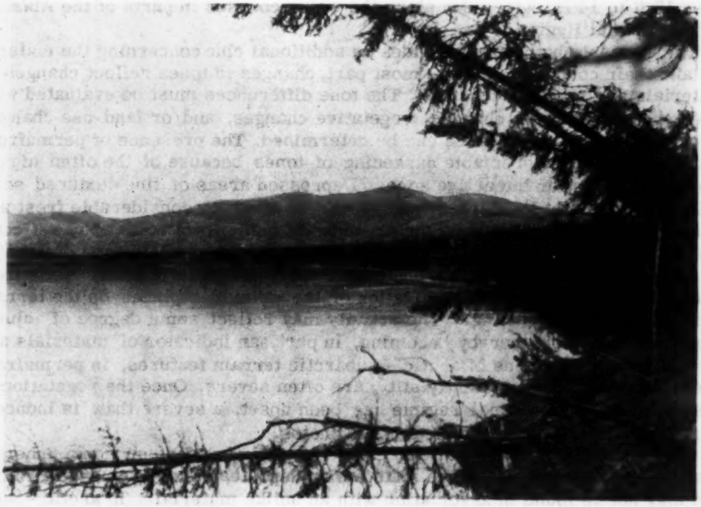

FIG. 23.—THAWING STREAM BANKS ARE OFTEN CHARACTERIZED BY LEANING ("DRUNKEN") TREES

FIG. 24.—BUTTON DRAINAGE IS ONE OF THE OUTSTANDING INDICATORS OF PERMANENTLY FROZEN SOILS

often 12 ft to 14 ft high, occur along the water courses in parts of the Alaskan Arctic Coastal Region.

The element photo tones provides an additional clue concerning the materials and their condition. For the most part, changes in tones reflect changes in materials and/or their condition. The tone differences must be evaluated with respect to topographic changes, vegetative changes, and/or land-use changes before their true significance can be determined. The presence of permafrost may result in an appreciable darkening of tones because of the often higher moisture content of the active zone. Depressed areas of fine-textured soils will usually reflect the darkest tones. Areas in which considerable frost activity occurs, as suggested by the presence of many frost boils, may reflect a very light tone if photos are taken after the surface materials have dried.

Cultural features on the airphotos are the reflections of man's activities. Their analysis leads to determination of the value man places on the terrain upon which he is operating. The activity may reflect some degree of adjustment to the terrain thereby becoming, in part, an indicator of materials and conditions. Alterations of arctic - subarctic terrain features, in permafrost areas which are predominantly silty, are often severe. Once the vegetation is destroyed and the thermal regime has been upset, a severe thaw is induced. This condition is easily detected on the photos.

Special features, as a pattern element, are dependent on some inherent quality or characteristic of the material or deposit. They are usually unique and may not be found in association with an unlike material. In arctic - subarctic - polar regions there are many features related solely to the permafrost regime and/or very severe frost activity, and if recognized they become an indicator characteristic.

DIRECT PERMAFROST AND FROST ACTIVITY INDICATORS

Recognition of many permafrost and severe frost activity features is often relatively simple provided that the photo scale and photo quality are such that the minute details are not lost.

In many permafrost areas polygons are among the more outstanding markings easily recognizable on photos of suitable quality and scale, with the exception of areas mantled with a dense forest cover (Figs. 24, 26, and 27). Polygons are direct indicators of the presence of permafrost. Polygons, in general, are confined to unconsolidated materials which have been rendered solid by freezing. In most instances the polygon indicates an undesirable soil situation—one in which there is a high percentage of ice in the soil mass. However, the photo pattern containing polygons must be evaluated in light of topographic position, soil parent material, and drainage since exceptions to the ice-soil relationship associated with the majority of polygons do occur. When they are found in association with silt soils it usually indicates ice either in wedge form or in massive layers depending on the type of polygon found. Polygons with raised centers and a continuous channel as an outlining perimeter contain ice in wedge form beneath the outlining channel. Polygons containing a depressed center and a perimeter consisting of a dike usually occur in association with fine-textured soils in depressed topographic situations. The center portion is usually underlayed by a layer of ice. It is the latter type of polygon which is perhaps the most critical when encountered in engineering construction.

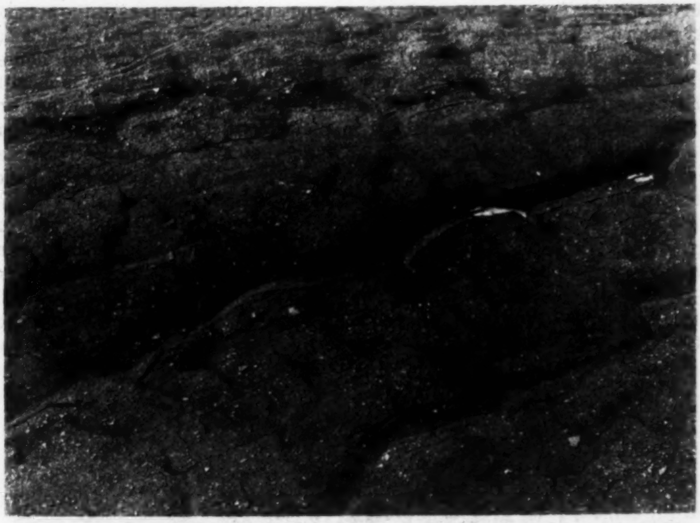

FIG. 25.—POLYGONS ARE AMONG THE POSITIVE INDICATORS OF PERMANENTLY FROZEN SOILS

FIG. 26.—DEPRESSED CENTER POLYGONS ARE ASSOCIATED WITH ONE OF THE MOST UNDESIRABLE SOIL-PERMAFROST RELATIONSHIPS

Frost mounds create another of the more outstanding features of the permafrost pattern. The largest ones, often called pingos, occur only in areas of coastal plain soils (Figs. 28 and 29). They are easily recognized by their dome-like or conical appearance and by their topographic situation, which is usually in the central portion of a basin or old lake bed. It is believed that they are the result of upheaval of the surface from subsurface ice pressure, which may reflect sources of water under great pressure below the permanently frozen materials. Many of them contain large cracks in the upper surface. It is not presently known if these reflect an artesian ground-water potential.

The various forms of soil movement related to frost activity and gravity on sloping ground create striking airphoto patterns. In some instances entire hillsides appear to be sliding down the slopes (Figs. 30 and 31). Surface markings include irregularly shaped flows, vegetation stripes extending down the slope, stone stripes, and great lobate or scalloped flows. In the arctic and subarctic the processes of solifluction are active agents of erosion and land destruction. The earth runs or flows consist of non-sorted material which is associated with frost activity and gravity. When a thick mantle of vegetation is present it tends to resist the downward movement until the load of frost-gravity moved rubble becomes too great and a rupture occurs. In areas where vegetation is sparse the soil flows usually extend unbroken for considerable distance down the slope. The channels between the flows appear darker because of higher moisture content and a vegetative cover. The very slight difference in elevation between the flow ridges and the intervening swales affords protection to the plants. Rocky slopes are marked with long streamers of rock slabs and fragments. The foregoing features are easily detected on good quality photos. These markings indicate a very undesirable frost condition—one which may result in severe heave during cold periods, exceptional instability during warm periods, and icing conditions during the winter months. Any use made

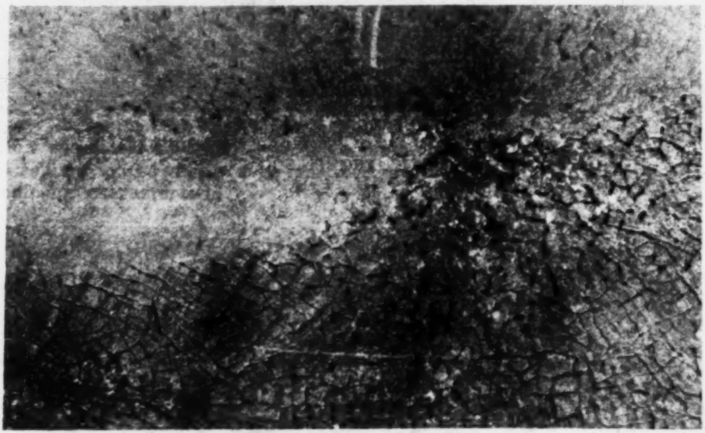

FIG. 27.—VERTICAL AIRPHOTO OF AN AREA CONTAINING RAISED CENTER TYPE POLYGONS

FIG. 28.—OBLIQUE PHOTO OF A PINGO IN THE ARCTIC COASTAL PLAIN

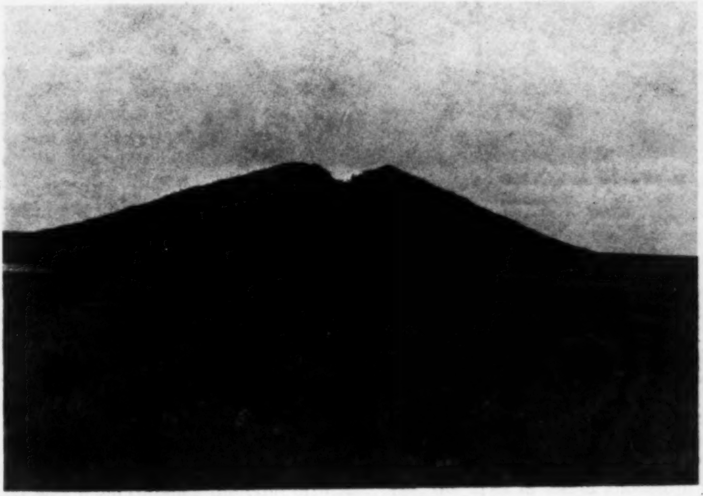

FIG. 29.—GROUND VIEW OF THE PINGO IN FIG. 28

FIG. 30.—OBLIQUE PHOTO OF A HILLSIDE MARKED WITH SEVERE SOLIFLUCTION LOBES

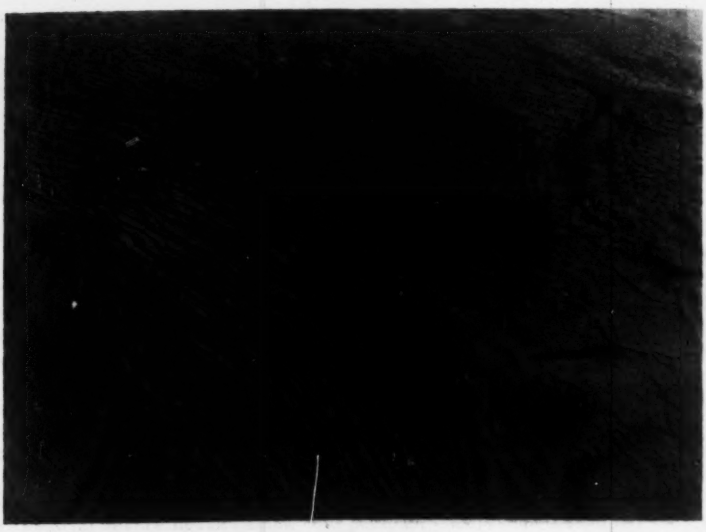

FIG. 31.—OBLIQUE PHOTO OF AN AREA IN WHICH DOWN HILL MOVEMENT OF SOILS (SOIL STRIPES) IS BURYING A POLYGON AREA

of hillsides marked with such features must take into consideration the severity of the problem.

Many arctic - subarctic polar areas are marked with minute mounds which are related to frost activity in the mantle or in the active layers (Figs. 32 and 33). In flat areas, the mounds are circular but tend to elongate with changes in slope. Mounds vary considerably in their size, shape, arrangement, and manner or origin. Some are believed to reflect the action of silt soil being forced upward through cracks or holes in the active layer and spreading out over the tundra vegetation. Many bouldery areas are marked with numerous small circular areas of fine-grained soil which appear to have worked up through the mass of boulders. In glacial till areas which are predominantly silty, the entire active zone appears to be constantly undergoing frost action turn over. Boulders appear to be working out of the ground in flat areas. In sloping areas, they are in process of burial from flow from above (Fig. 34). In areas swept by high winds the silty soils are carried away as soon as they reach the surface and dry out, thus, leaving a surface which resembles a "desert pavement." Such situations are misleading as the impression of great granularity is created (Fig. 35).

TYPICAL ANALYSIS

One area in Alaska has been selected for purposes of illustrating the airphoto method of obtaining information about earth surface features. It is illustrated in Fig. 36.

Area I.—Aside from location, the regional environmental factors were obtained from study of a large number of photos assembled into mosaic form. The study area is situated in Northern Alaska, north of the Brooks Range and just west of the big bend of the Colville River. The study area is crossed by the Colville River. From the physiographic standpoint the entire study area lies in an upland plateau which is situated between the mountains to the south and a vast coastal plain to the north. The area is often referred to as "Arctic Piedmont" or "Arctic Plateau." The upland is characterized by gently rolling topography which is everywhere rounded except along the Colville River. A decided east-west trend of major forms is in evidence. The Colville, as it crosses the area, has cut a rather deep valley in the upland. In places, steep-sided bluffs rise several hundred feet from the valley floor. The valley is quite broad in places and contains several terraces situated between the flood plain and the upland proper. The terraces are low topographically and exhibit very low relief.

From a geologic standpoint the area appears to be composed of gently dipping strata of interbedded hard and soft rock which is overlain by an unrelated mantle of varying thickness. It appears that the softer materials occur in the thicker beds. An occasional prominent ridge provides a significant break to the generally rolling landscape. The structure generally strikes east-west. The presence of the unrelated mantle is suggested by the over all softening of upland slopes and by drastic changes in slopes associated with intense erosion.

From the climatic standpoint, the area exists under generally low rainfall conditions in an environment of cold winters, and short, but relatively warm, summers. There are ample indicators which suggest the presence of permafrost which establishes the cold climate. Yet, the area is warm enough in the summer to support some variety of vegetation.

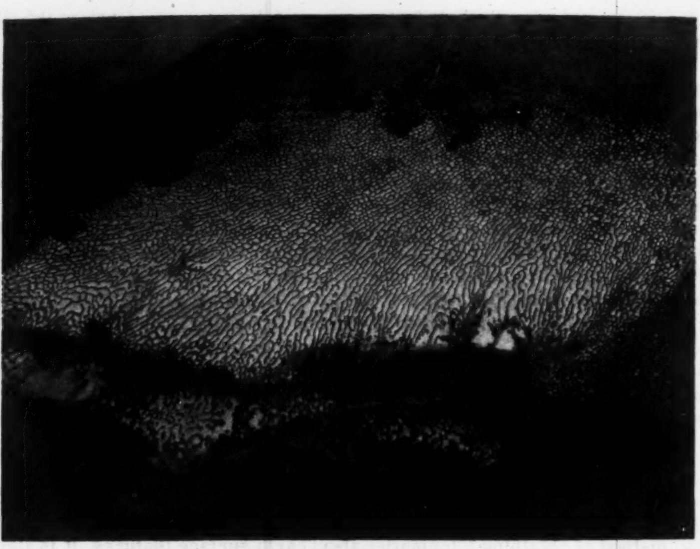

FIG. 32.—OBLIQUE PHOTO OF A HILLSIDE MARKED WITH FROST BOILS AND SOIL STRIPES

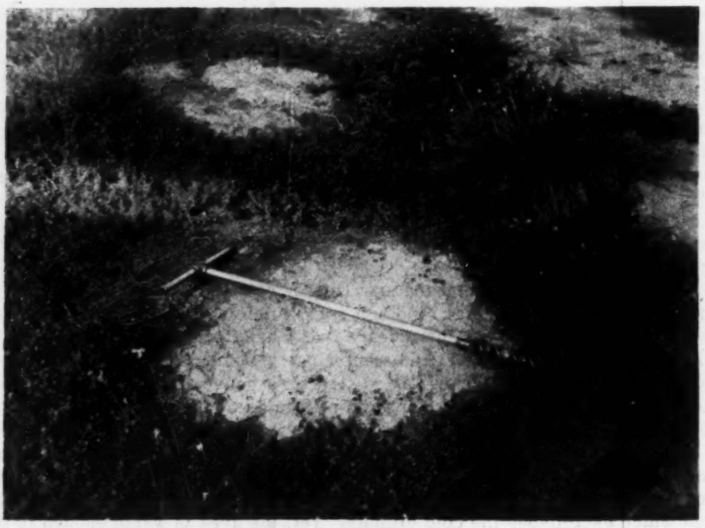

FIG. 33.—DETAIL OF SMALL FROST BOILS IN A SILTY SOIL MANTLE

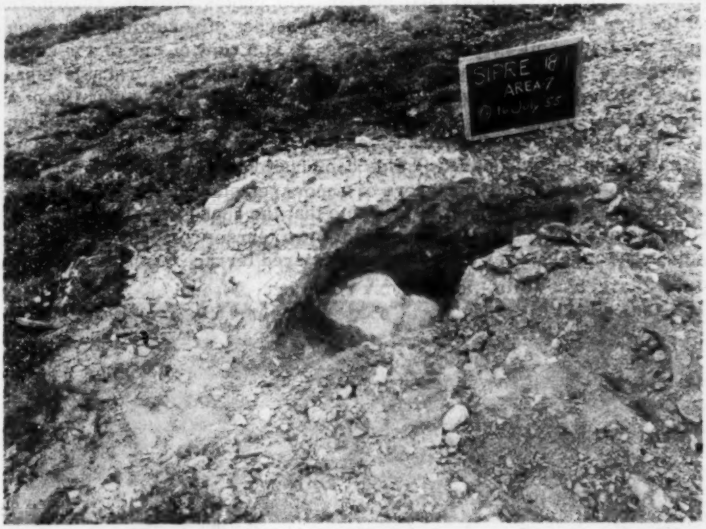

FIG. 34.—LARGE BOULDER BURIED BY SILTY SOIL WHICH HAS MOVED FROM ABOVE THROUGH FROST ACTION-GRAVITY PROCESSES

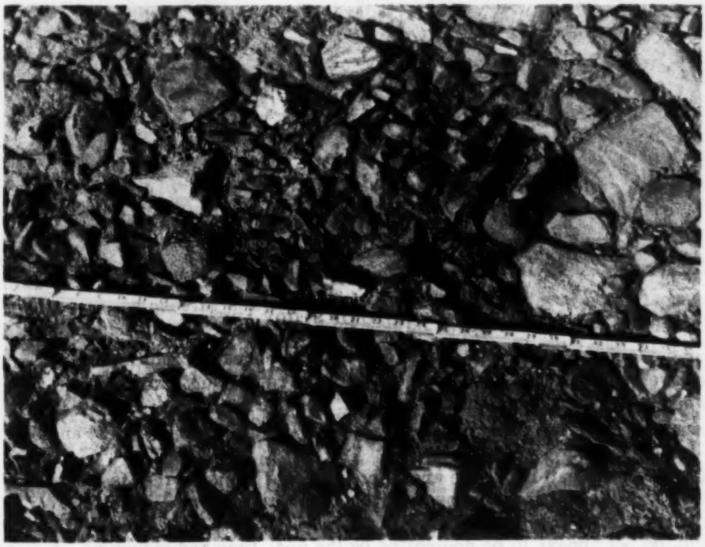

FIG. 35.—REMOVAL OF FINE-TEXTURED SOILS BY HIGH WINDS CREATES A "GRAVEL PAVEMENT" SURFACE

Detailed study of the individual stereoscopic photos provides information about the type of materials in the area, the presence of permafrost, and some concept about ice-soil relationships. Fig. 36 is a copy of one of the contact prints which has been marked to show significant features. First, the area of the single photo has been divided into three major landforms—upland, terrace and flood plain. The upland consists of a portion of a dissected highland (plateau) having, generally, softly rounded slopes. The slopes are broken in places by prominent rock outcrops (R). This combination of features suggests thick layers of soft material and thin layers of a more resistant material. Since there is a repetitive lineal pattern it is probable that the rocks are slightly dipping. The terrace area appears to be composed of two terrace surfaces, one of which is quite broad and the other occurring as scattered terrace remnants (TR). The upper surfaces are not at the same elevation. Boundary conditions of the terrace have been difficult to locate because of the rather subtle topographic break between terrace and flood plain in places. Separation has been based on other pattern features. The flood plain contains the stream, islands, bars, and other low-lying areas.

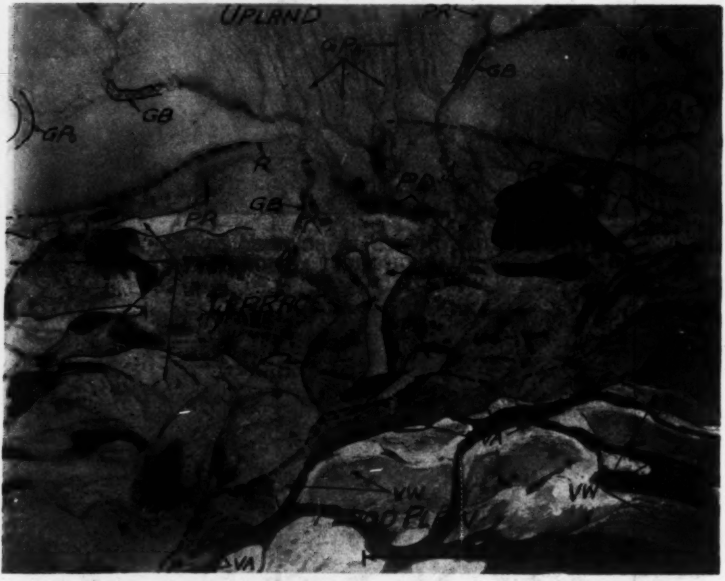

FIG. 36.—AIRPHOTO OF STUDY AREA

The drainage pattern of the area, on a regional basis, is trellis in plan suggesting that the area is composed of layered hard and soft tilted rocks. The second and third order streams, and those which have not cut deeply into the upland, do not appear to be controlled by the rock type and structure. A more random arrangement exists. This suggests the presence of an unrelated mantle in the rocks. As the upland streams cut through the mantle and reach the rock below, a change in plan and stream profile occurs.

The erosional features reflect the permanently frozen condition of the materials. Upland gullies include placer-type gullies lined with intense polygon nets (GPo), parallel gully systems (GPa), and button type gullies (GB). The parallel gullies discharge on to the terrace and do not continue across the terrace. The button gullies also discharge on to the terrace but extend for a considerable distance outward. The circular pools are larger on the terrace than in the upland position of the same gully system. There is no well-developed surface drainage system on either the flood plain or the terrace.

Tundra-type vegetation prevails throughout. There are no indications that barren conditions prevail. It is probable that the terrace and upland supports grasses, berries, mosses, and dwarf varieties of willow and birch. Larger plants occur along the water courses. Willow, perhaps 4 ft to 5 ft high (Vw), occurs on the better drained granular soils of the islands and bars. Alder thickets (V_A) occur on cold wet soils in the flood plain.

The photo tones of the entire area are, generally, light. Open-water areas are black. Some of the, generally, wet areas are dark grey. Tones, in this instance, are related more to the predominance of light green of the vegetation than to the actual color of the mineral soils.

There are many special or unique features which are in evidence and which lend strong support to permafrost and ice-soil relationship. Permafrost features in the upland include button-type gullies (GB), polygon lined placer gullies (GPo), "geometric" (those whose thread follows a series of polygon channels) gullies (GG), large thermokarst lakes (LT), areas of depressed-center polygons (PD), area of raised-center polygons (PR), and thawing polygon channels (PT). Permafrost features on the terrace include extensive areas of depressed-center polygons (PD), extensive areas of raised-center polygons (PR), button-type gullies on the terrace remnants (GB), large rectangular polygons on higher and more granular parts of the terrace (PR_e), and numerous abandoned meander lakes which are being obliterated by vegetation invasion. Thus, the upland mantle can be expected to be frozen within perhaps 12 in. to 18 in. of the surface. In areas protected by thick moss the active zone may be within 6 in. to 12 in. On the terrace, the light-toned terrace remnants can be expected to be unfrozen for 4 ft to 5 ft during the latter part of the thaw season. One type of adverse soil permafrost conditions are associated with the depressed-center polygons (PD) as the major portion of the upper few feet will contain much ice. Another is that related to channel type polygons (PR) in which ice will occur in wedge form beneath the channels.

From an engineering standpoint the upland soils consist of a semi-granular to non-granular mantle of varying thickness on sandstone and shale. The mantle is shallow in areas having prominent topographic breaks. Local valleys are filled with colluvial soils which are, generally, fine-grained. The most serious permafrost conditions occur on sloping ground, in the upland depressed situations, and on polygon ground. Shallow cuts are to be avoided as they will not intersect rock and severe thaw will occur. Deep cuts which extend to the sandstone and utilize the sandstone for support will be permissible but will be difficult to do because of the disturbance of the surroundings. Runway orientation which is coincident with the long sandstone ridges will offer one feasible solution. All valley-fill type gullies, depressions, and side-hill situations will require fill and should not be disturbed prior to use.

The higher portions of the terrace (TR) consist of frozen sand and gravel with scattered areas of fine-grained soils on the surface. Limited amounts of granular materials are available for borrow when the seasonal thaw has reached a maximum. The remainder of the terrace is low and marshy and contains considerable subsurface ice. The soils are predominantly fine-grained with considerable peat in the upper part. The terrace remnants will offer good sites for buildings. In this location, they are too small for use in runway location. The remainder of the terrace area should not be disturbed. If it is necessary to utilize any portion then end-dump-fill procedures should be followed. The flood-plain soils are granular and contain materials available for construction use.

SUMMARY AND CONCLUSIONS

In concluding this discussion on the use of aerial photographs for obtaining information about earth-surface materials and their condition in arctic - subarctic regions, it is well to point out a few of the more important results and conclusions concerning the present stage of development.

1. Accurate determinations of soils and permafrost conditions employing photo-interpretation techniques is contingent on a background in some of the earth sciences as well as some phases of civil engineering, experience in the arctic and subarctic, cognizance of the relationship between the areal soil patterns and engineering problems, a keen imagination, and the ability to apply processes of logic and deductive reason to the analysis of physical features as reflected in the airphotos.
2. Proficiency in using airphoto analysis techniques makes it possible to distinguish good materials and good site areas from detrimentally frozen materials and unsatisfactory site areas.
3. The presence, degree, and type of permafrost, as well as areas of severe frost susceptibility, can be predicted in most situations in the arctic and subarctic.
4. The general physical characteristics of soil-parent materials as well as materials for engineering construction can be identified and delineated with east in most regions throughout the arctic and subarctic.
5. It is possible to make general determinations concerning whether a material is chiefly sand, gravel, silt, or clay and, in some instances, it is possible to predict combinations of the foregoing. Detailed information about such physical properties as grain size, structure, engineering limits, profile development, and natural densities cannot be obtained by direct photo analysis methods.
6. It is not possible to determine the suitability of granular material for aggregate for Portland cement concrete or bituminous concrete use.
7. In many instances it is possible to predict the presence and type of overburden on granular materials which may be considered for construction use.
8. In areas where rock is being considered for use as a source of engineering materials the following information can often be obtained from airphotos: lithological and structural characteristics; degree and type of weathering and presence of an overburden; landslide potential; trafficability condition of both the site and access routes to the site; and presence of talus materials.

9. When talus material is being considered, proper use of the method will often make it possible to distinguish between those materials which are dry and unfrozen and can be excavated, and those materials which contain considerable fines in their mass and which are frozen and cannot be excavated by normal procedures.

10. It is not possible to predict the amount of material, either rock or granular, expected to be obtained from a quarry or pit.

11. From a trafficability standpoint, the trafficable and non-trafficable areas can be identified and delineated for heavy construction equipment use.

12. In undeveloped and unmapped regions airphotos can be used to great advantage in all activities pertaining to airfields, highways, and other engineering works by making general engineering soils and drainage maps which show good and bad soils based on anticipated performance.

13. A materials survey effected through the use of airphotos can be done more rapidly, more efficiently, more accurately, and at a greater savings in cost than by use of any other method.

14. The principles as developed in the temperate climates have found considerable application in the arctic and subarctic and it is believed that they can be applied to any other region, regardless of climate, where engineering studies involving the natural and physical features are concerned.

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TRANSACTIONS

Paper No. 3154

EVALUATION OF ALTERNATIVE SUBWAY ROUTES

By B. A. Griffith[1] and H. G. von Cube[2]

SYNOPSIS

A practical automatic computer method of assigning passenger trips to a proposed traffic facility is outlined and the report gives the results of applying this method to proposed subway systems in Metropolitan Toronto.

Where adequate origin-destination data are available, the method can be used directly for a rapid transit system, or with minor modifications for other types of traffic facilities, such as commuter lines or expressways.

It is obvious that when $200,000,000 are to be spent on a facility such as a subway, it is most desirable to ensure that the maximum benefits will be derived. Authorized by the Metropolitan Toronto Council to report on the most desirable location of an east-west subway, the Metropolitan Toronto Planning Board faced the problem: How could the merits of any number of hypothetical transit lines be evaluated?

Besides thorough studies of the engineering feasibility in construction and operation, studies of the economics of the situation, and the future development of the city, there was the problem: What usage could be expected of each of the hypothetical rapid transit facilities? To solve this problem, the writers' firm was contracted to work on the problem with the objective of predicting passenger volume and behavior on various alternative new subway lines proposed for Metropolitan Toronto.

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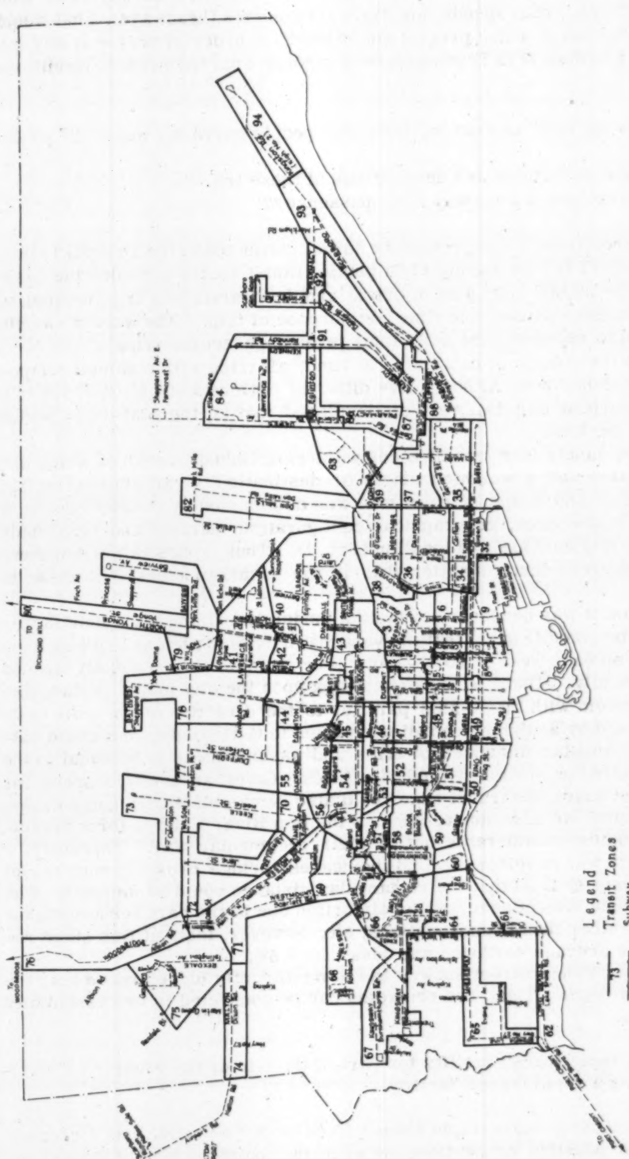

Legend
Transit Zones
Subway
Street Cars, Buses & Trolley Coaches
Metropolitan Area
TO RICHMOND HILL
TO WOODBRIDGE
MALTON AIRPORT

FIG. 1.—PUBLIC TRANSIT SYSTEM AND TRANSIT ZONES IN METROPOLITAN TORONTO.

Thus the problem may be formulated as follows: Given a subway route with specified stations, train speeds, and train frequencies (headways), what would be the expected usage under present conditions? In order to arrive at any answer to this problem, it is first necessary to know what the present conditions are, that is,

1. What is the total number of trips that people currently make via public transit?
2. What are the origins and destinations of these trips?
3. How attractive is a subway for a passenger?

Data obtained from a comprehensive survey conducted by the Toronto Transit Commission (TTC) on twenty eight public transit routes provided the basic information for 50,000 riders as to origin and destination of a trip, number of routes and transfer points, and time and purpose of trip. The survey sample was expanded to represent the daily total revenue passenger trips.

To reduce the volume of data cards at hand, all trips with common components were summarized. Although five different periods were allowed for in a day, for our purpose only two were distinguished, that is, the rush-hour period and all other periods.

In addition, ninety four transit zones were established in each of which the centroid represented a weighted origin or destination for all stations in that zone. The size and shape of each zone was determined by considerations of trip density in the area, geographical configuration, natural and man-made obstructions, and land use and location (Fig. 1). Thus, zones in the downtown business area were much smaller than in the suburban areas (not shown in Fig. 1).

At this point it was necessary to build a model describing all transit facilities. It will be noted (Fig. 1) that the vast majority of streetcar and bus lines are operated on east-west or north-south lines somewhat irregularly spaced throughout the city. Thus, by superimposing upon the city map a rectangular coordinate system with the X-axis parallel to the direction of the main east-west streets and by assigning coordinate values to O-D points, one could calculate the rectangular distances of trips and assume these to be equal to the distance travelled on surface transit facilities. Given an average speed for streetcars and buses, the travel time could also be calculated. In some cases, barriers, natural or man-made, such as the Don River Valley, force travelers to use routes considerably out of their rectangular way. Therefore, a minor program was developed to find the nearest bridge crossing such travel barriers between O-D so that the rectangular distance could be adjusted. One may expect some loss of accuracy or distortion due to the card reductions, the use of zones rather than transit stations, etc; however, tests at this stage revealed that the average deviation was less than 2.5%.

The existing Yonge Street subway has a significantly higher speed than the surface transit lines. Thus, two centroids can be connected in two essentially different ways:

a. Using a rapid transit facility for part of the trip or the whole.
b. Not using a rapid transit facility.

By superimposing more rapid transit lines on our network, one increases the number of possible connections between two points using the subway for part or the whole trip. The computer program developed, handled up to ninety possible subway connections between two centroids. Such a subway system may have six junctions or interesections.

The travel distance from any origin to any destination and using the rapid transit facility then consisted of:

1. Access distance: that is, travel to and from the nearest subway station by surface routes.
2. Rapid transit distance: that is, travel from entry to exit station on rapid transit.

In order to allow for the difference in speed of travel between travel on rapid transit and travel on surface routes, the rapid transit distance was converted into equivalent surface distance according to the various speeds set for any of the proposed subways. In addition, allowances were made for transfer times according to the frequency of trains or streetcars and buses.

After computing the time and distance of travel on all alternative routes available using rapid transit for all or part of the trip, the best route was selected and this compared with the time and distance of travel exclusively by surface transit, thus obtaining time and distance lost or saved. In practice, however, not all transit passengers having identical origins and destinations use the route calculated to be the most efficient.

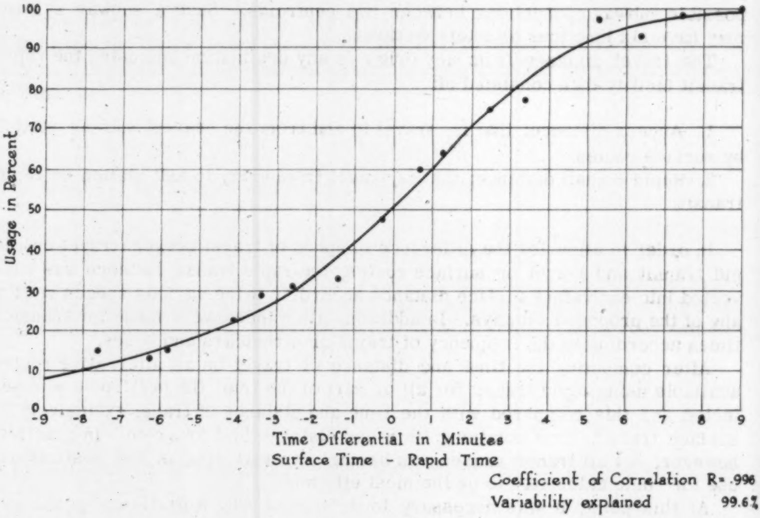
At this point, it was necessary to determine why a particular passenger chooses one travel route in preference to an available alternative. Here, one is really concerned with motivation. It seems likely that several factors may influence him—convenience, comfort, time, distance, companionship, scenery, etc. Of these, only time and distance are readily measured for each alternative route.

To determine the number of passengers who would use a rapid transit route in preference to a surface route, a study was made of passengers using the Yonge Street subway to determine their time and distance benefits compared to travel by the surface route. For all trips between the same O-D, it was known which routes were actually chosen and so it was possible to prepare a tabulation showing the percentage of individuals who do, in fact, choose to use the Yonge Street subway. This percentage was plotted against time (or distance) saved and smooth curve was drawn through the points obtained (Fig. 2). The scatter of points about this curve was found to be considerably less for time than for distance. Such a curve represents what is known in economics as the indifference in choosing between two alternatives: that is, an indifference curve. Due perhaps to improved surface facilities during rush hour periods, the curve for those periods is somewhat different than for the remainder of a normal week-day (Fig. 3).

Before these indifference curves were developed, the estimated number of rapid transit passengers on a hypothetical transit system was obtained by assuming that 100% of all passengers not losing time would use rapid transit. To determine the difference between the old and new method, a comparison was made. The assignment based on an indifference curve produced an increase on the average of 8% and a more realistic passenger flow.

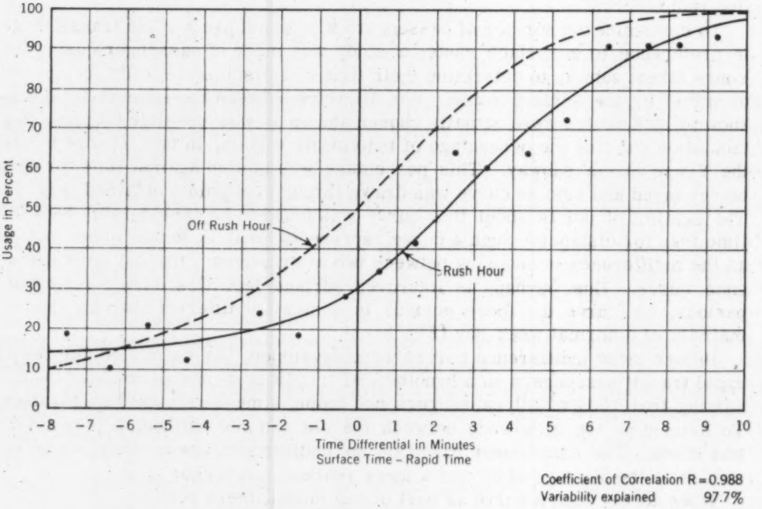
If we accept such a curve as part of our model, there result

(a) Demands for trips between centroids.
(b) Travel times and distances for all trips on alternative routes.



Coefficient of Correlation R= .996
Variability explained 99.6%

FIG. 2.—PERCENTAGE USAGE VS. TIME DIFFERENTIAL IN OFF-RUSH-HOUR PERIOD



Coefficient of Correlation R = 0.988
Variability explained 97.7%

FIG. 3.—PERCENTAGE USAGE VS. TIME DIFFERENTIAL IN RUSH HOUR PERIODS (FOR COMPARISON THE OFF RUSH HOUR RELATIONSHIP IS ALSO SHOWN)

(c) An indifference curve for determining the preferred route among several alternatives.

Thus the entire computer program consisted of:

(i) Selection of the best transit route with or without the use of a subway.
(ii) Assignment of passengers to the selected route.

The output of this program gave the expected passengers boarding and leaving the subway at each station along the proposed route, as well as the volume carried between stations. In detail the following information was produced:

1. Total daily passenger trips assigned to each assumed facility.
2. The number of passengers boarding and alighting at each rapid transit station.
3. The passenger volumes between each station on the line.
4. The origin and destination of trips in relation to the transit station of origin and destination used.
5. The time lost or saved by using the rapid transit system for all or part of the trip in preference to using surface transit for the entire trip.
6. The distance lost or saved by using the rapid transit system for all or part of the trip in preference to using surface transit for the entire trip.
7. The division of total trip length by distance on rapid transit and distance on surface transit.
8. The division of the above data by 'rush' and 'off-rush' travel periods of the day.
9. The summary of the number of passengers shown to use the rapid transit system by units of time and distance lost and/or saved.
10. Number of transfers made for each system for each O-D trip.

Analysis of the assignment results permits a detailed appraisal of each hypothetical rapid transit system and a comparison of the systems as follows:

a. The total passenger trips on each system by times of travel.
b. The distribution of trips on each line.
c. The comparison of trip length on rapid transit to total trip length.
d. The comparison of trip time on rapid transit to surface transit to total transit time.
e. The effect of a rapid transit system on existing surface lines and hence the possible savings on those lines resulting from rescheduling.

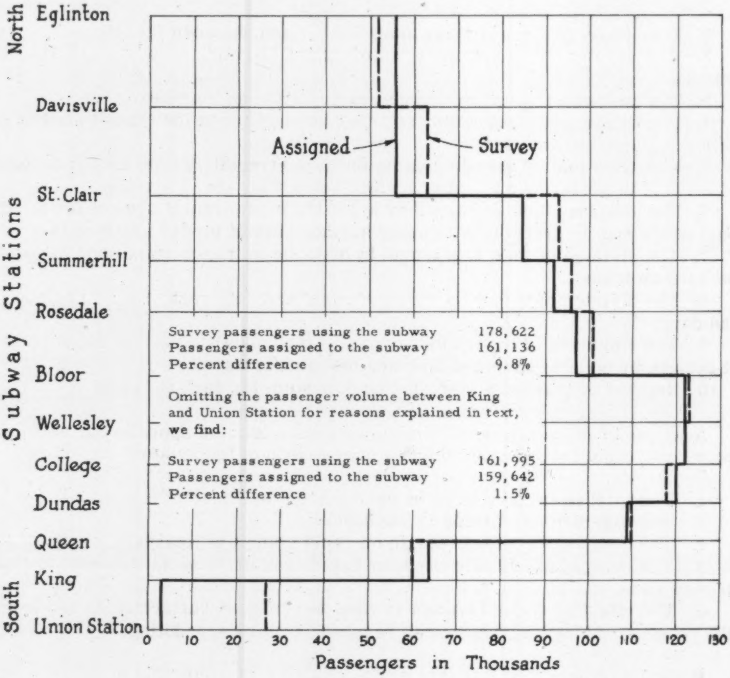
It should be expected that this model applied to a prediction of the passenger volume of existing systems should produce, within certain error limits, the existing passenger volume on the line.

In order to test the model it was applied to:

1. The present Yonge Street subway (Fig. 4).
2. The east-west streetcar line on Bloor Street regarded as a subway with speeds and headways conforming to those presently in use on that line.

In both cases, the travel pattern was reproduced to a high degree of accuracy. For one or two sections of these lines, zoning effects produced discrepancies much larger than those found at all other sections. For example, the center of one downtown zone was so located that the majority of trips from that zone used surface routes to reach the Yonge Street subway, whereas in fact, many of them would and do board the Yonge subway at Union Station. By splitting such a zone to yield two centers, one near Queen Street, the other near the Union Station, this difficulty could be overcome.

Discounting the discrepancy between predicted and existing passenger volume at the Union Station, the difference, in percentage between the predicted and the existing passenger volumes is 1.5%. The difference (as a percentage of the total passenger volume including the Union Station) is 9.8% and lies within error limits. Thus, for the purpose of prediction the principles developed proved to be very satisfactory.



Survey passengers using the subway	178,622
Passengers assigned to the subway	161,136
Percent difference	9.8%

Omitting the passenger volume between King and Union Station for reasons explained in text, we find:

Survey passengers using the subway	161,995
Passengers assigned to the subway	159,642
Percent difference	1.5%

FIG. 4.—ASSIGNED AND SURVEY PASSENGER VOLUME ON YONGE SUBWAY

The model which has been described was used to determine the expected 'present' usage of the proposed Bloor-University line[a] which the TTC plan to built. It was also used to test one or two other proposed subway routes. One of these, the so-called U-line[b] (of which two versions, U_1 and U_2, were considered) gave a slightly better expected total usage than the route planned by the TTC and also gave considerably more relief to the overcrowding on the present Yonge Street line (Figs. 5 and 6). The principle objection to the U-line is its failure to join the east and west parts of Bloor Street by a direct subway line, thus removing surface traffic from the congested central part of Bloor Street. It would of course give some indirect relief to this central section of Bloor Street; that this indirect relief would be sufficient, has been seriously questioned.

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[a] The Bloor-University line runs east-west on Bloor St. and Danforth Ave., with a spur southward on University Ave. to join the Yonge St. subway at Union Sta. This line is also referred to as the 'Wilson line' (cf. Fig. 6), after the design engineer, Mr. Norman D. Wilson.

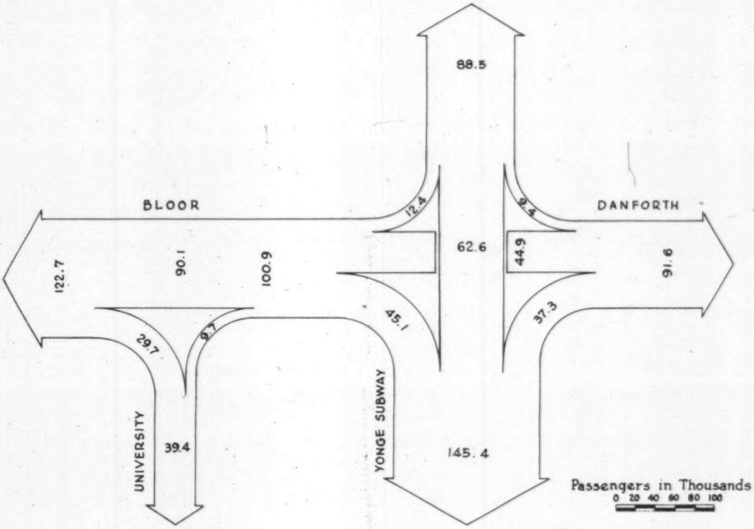

FIG. 5.—INTERCHANGE OF PASSENGERS ON THE PROPOSED BLOOR–UNIVERSITY SUBWAY, AT INTERSECTIONS OF YONGE AND BLOOR AND AT UNIVERSITY AND BLOOR

CONCLUSIONS

The model which was presented herein can be adapted to tests of future subway proposals, to tests of commuter routes on rail lines, and (with minor modifications) to assignments of traffic to expressways. In the future, it is hoped that the application of such a model will be considered in conjunction with engineering studies, and that such model studies will have the whole-hearted support of all interested parties. In this way it will be possible to build into the

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[b] The U-line runs for a short distance on Bloor St. West, turns south to Queen St., proceeds east on Queen St., and later turns north to continue a short distance east on Danforth Ave. (see Fig. 1).

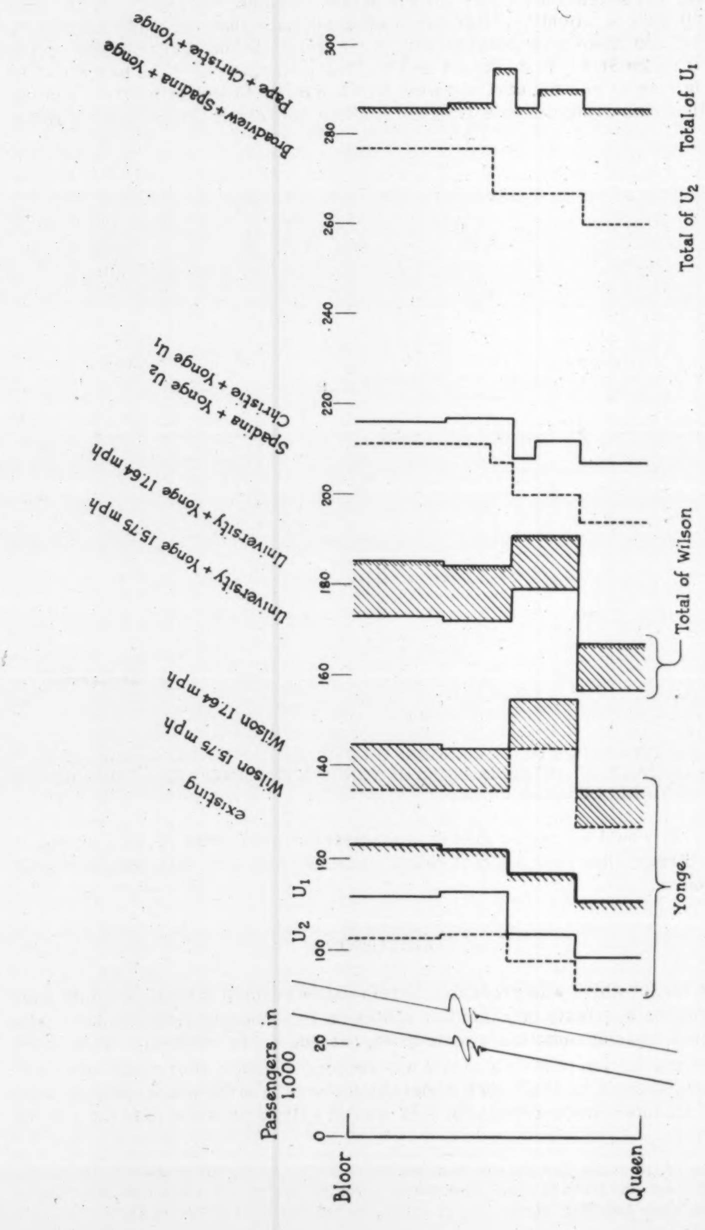

FIG. 6.—NORTH-SOUTH MOVEMENT OF PASSENGERS ACCUMULATED FOR BLOOR-UNIVERSITY, U_1 AND U_2 RAPID TRANSIT SYSTEMS

model realistic descriptions of details such as speeds, stations, headways and transfer times for subways and commuter lines or road widths, speeds, and interchange facilities for expressways.

APPENDIX.—REFERENCES

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2. "Measuring the Potential Traffic of a Proposed Vehicular Crossing," by Nathan Cherniak, Transactions, ASCE, Vol. 106, 1941.
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EPOXY ASPHALT CONCRETE FOR AIRFIELD PAVEMENTS

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SYNOPSIS

Studies of airfield-pavement problems which arise from the combined effects of heavy load, fuel spillage, and jet blast have led to the development of a new type of paving material called epoxy asphalt concrete (EAC). Conventional hot-mix asphalt plants and paving equipment are used in its production.

This material has Marshall stability values of 15,000 lb to 20,000 lb and is capable of withstanding tire pressures in excess of 1,000 psi. In repeated gyratory loading simulating B-52 traffic, EAC resists densification and retains its load-carrying ability. Good retention of strength and stability under severe conditions of solvent and fuel spillage has been established by experience in aircraft maintenance areas. In jet-blast tests with military planes, EAC performs well under normal pretakeoff conditions and with prolonged afterburner operation producing pavement temperatures of 800°F.

Investigation of its mechanical properties shows that EAC has a flexural strength or modulus of rupture exceeding that of portland cement concrete by a factor of 3 to 6. However, EAC has flexibility which allows it to be bent to the same extent as asphaltic concrete before fracture. The new paving material thus combines the strength of portland cement concrete with the flexibility of asphaltic concrete.

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INTRODUCTION

The rapid growth of both military and commercial aviation has been accompanied by ever-increasing aircraft weight, higher wheel loads, and higher tire pressures which have posed many structural problems in the design and construction of adequate pavements. To these structural problems have also been added some special requirements which have accompanied the development of jet aircraft: these are good resistance to the high temperatures and erosive effects of jet blast, and the ability to tolerate fuel spillage. The nature of these problems has been investigated by a number of military and commercial organizations and has been well presented at the Jet Age Airport Conference held in New York, N. Y.[5] Thorough discussion of these problems has also taken place at three United States Air Force Pavement Conferences[6] which have brought together representatives of military and civilian aviation groups, military personnel responsible for the design and construction of airfields, and representatives of various paving industry groups. From a study of the papers presented at these conferences it is clear that neither rigid nor flexible pavements of the conventional type offer a complete solution to the combined problems of very heavy load, high temperature, high velocity blast, and fuel spillage. Any new paving material which can provide performance under these conditions which is superior to that obtainable with conventional pavements should find a place in airfield pavements of the future.

A NEW PAVING MATERIAL

From research on a large number of new paving compositions, epoxy asphalt concrete (EAC) has emerged as a material well suited to solve simultaneously the problems of heavy loads, high temperature and velocity blast, and fuel spillage. It is a combination of a graded mineral aggregate and EPON* asphalt binder. The properties of the pavement are determined by the properties of the binder which contains epoxy resin well-known for its outstanding performance as an adhesive and for its remarkable physical properties. Initially the binder is a black, viscous liquid hardly distinguishable from asphalt. During mixing with hot aggregate, subsequent hauling, laying, compaction, and field curing, the binder is converted by a chemical reaction from a viscous liquid into a non-melting elastic polymer with interesting mechanical properties and solvent resistance. The rate of the polymerization reaction taking place is readily controlled so that the aggregate-binder system can be handled in normal hot-mix asphalt plants and can be spread and compacted by conventional paving machines and rollers.

The finished pavement has the appearance of normal asphalt concrete but the properties are very different in many respects. Initial applications of epoxy asphalt concrete have been in the form of overlay pavements $\frac{1}{2}$ in. to 1 in. thick which have been used as a protective layer for existing flexible and rigid pavements. Studies are under way which should lead to a design for the use of epoxy asphalt concrete as the major pavement element in new construction.

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[5] Proceedings, ASCE, Vol. 83, No. AT2, December, 1957.
[6] Roads and Streets, September, 1959, p. 111.
* Trademark, Shell Oil Co.

EPOXY ASPHALT BINDER

The binder is composed of a liquid epoxy resin and a paving grade asphalt containing an additive which acts as a flexibilizing coreactant. When these ingredients are mixed in the proper proportions, a chemical reaction begins which causes the viscosity to rise steadily until the binder is converted into a thermoset plastic. The rate of the reaction and the corresponding rise in viscosity are determined by the temperature of the system. The reaction may be caused to proceed at a rapid rate by choosing a high temperature or at a slow rate by choosing a low temperature. The operating temperature range for practical handling of the binder in paving applications is about 200°F to 300°F and includes the preferred operating range of hot mix paving plants.

When the binder is cured in the form of sheets it may be tested in tension according to ASTM method D412-51T. Dumbbell-shaped specimens stamped from the sheet and tested at a rate of 20 in. per min at room temperature show a tensile strength of about 1,000 psi and an elongation at break of 200% to 300%. Correcting the tensile-test value for the change in cross-sectional area during the test gives a true tensile stress at fracture of about 3,000 psi. The binder is elastic and will return to its original shape after deformation by twisting or stretching.

The binder is particularly well suited to high-temperature use as shown by the fact that it does not melt or lose its shape when heated to 800°F on a hot plate. Heating to a high temperature does not destroy the good low temperature flexibility of the binder as indicated by the results of the Fraass test shown in Table 1. In this test (Institute of Petroleum method 80/53) a 0.5 mm thick layer of asphalt or other product is placed on a spring-steel strip and subjected to periodic bending which produces a maximum tensile strain of 0.03 in. per in. over a period of 11 sec. The temperature is gradually lowered until the specimen fractures under this strain. Table 1 shows that a 60 penetration paving asphalt fractured at +16°F in this test while epoxy asphalt binder failed to fracture at -31°F. After exposure to a temperature of 760°F for 5 min the Fraass breaking point of the asphalt was raised to +25°F while the epoxy asphalt still failed to fracture at -31°F.

TABLE 1.—LOW TEMPERATURE FLEXIBILITY OF EPOXY ASPHALT AS SHOWN BY THE FRAASS TEST

	Initial	After 5 Min at 760°F
60 Pen Asphalt	+16°F	+25°F
60 Pen Epoxy Asphalt	-31°F	-31°F

PREPARATION AND PLACEMENT OF MIXES

Epoxy asphalt concrete is prepared in conventional hot-mix asphalt plants, laid with self-propelled paving machines or by hand spreading, and compacted with conventional steel and rubber-tired rollers. Construction specifications for successful application of this new product have been established in a series of field installations conducted during the past 3 yr using commercial hot-mix plants and paving equipment.

Mix Plant Operation.—Minor additions to the mix plant are required to provide for pumping of the epoxy resin and additive to the weigh platform where blending of the binder components is accomplished in the weigh bucket with brief stirring. Alternatively, the additive is preblended with the asphalt and only the epoxy resin is added in the weigh bucket. Laboratory and field studies of mix design have shown 8% to 10% weight of binder basis aggregate to be the preferred range for preparation of dense, flexible, and impermeable epoxy asphalt concrete. Asphaltic concrete mixes containing 8% to 10% binder would have very poor stability at the high pavement temperatures encountered in warm climates and under jet blast because the asphalt is essentially a viscous liquid at high temperatures. Since epoxy asphalt is a non-melting plastic which retains its shape at high temperatures it is possible to obtain flexible mixes with high binder contents without the serious loss of stability usually experienced with asphalt.

Aggregate complying with conventional quality and gradation specifications is used, and for overlay pavements $\frac{3}{4}$ in. to 1 in. in thickness, the maximum aggregate size is set at $\frac{1}{4}$ in. Dense graded aggregates are preferred where dense, impermeable pavements are desired to act as a protective overlay for asphaltic concrete against attack by various fluids. Aggregate temperature is an important item in successful handling of epoxy asphalt concrete since the rate of the chemical reaction taking place in the binder, and hence the binder viscosity, is determined by the temperature of the mix.

Timing the Paving Operations.—The mix temperature has an important influence on the time available for subsequent operations involved in laying the pavement. At high temperatures the polymerization reaction proceeds more rapidly, and the viscosity of the binder increases more rapidly than at low temperatures. The mixing of binder and aggregate presents no problem because the viscosity of the binder remains for about 30 min within the viscosity range in which penetration grade asphalts are normally mixed with aggregate. Normal mixing time in the pugmill is required, usually 1 min or less. Laboratory and field experience have shown that the mixing, hauling, and spreading of the mix should be accomplished by the time the binder viscosity has reached 60 poises in order to be able to obtain the desired degree of compaction with conventional rollers. The aggregate temperature to be used in any given construction job will be determined ordinarily by the hauling time involved, since the mixing time in the plant and the time required for spreading are essentially constants. Where the hauling time is short, relatively high aggregate temperatures can be used, but for long hauls lower temperatures are required.

Surface Preparation.—When epoxy asphalt concrete is used as an overlay on existing pavements, the surface should be swept free of dust and dirt and a suitable tack coat should be applied. Tack coats of emulsified asphalt, cutback asphalt, and penetration grade asphalt have been used successfully in many installations. Where the pavement is subjected to normal pretakeoff conditions by military or commercial jet aircraft, an asphalt tack coat appears adequate. In some cases where the overlay is placed on portland cement concrete and is subjected to prolonged high-temperature blast, such as is maintenance and engine overhaul areas, a tack coat of epoxy asphalt binder has been successfully used.

Compacting the Mix.—Laboratory studies and examination of field installations have shown that low void content of the compacted mix is very beneficial in obtaining the impermeability and high solvent resistance desired in epoxy asphalt concrete overlay pavements. The retention of strength after long exposure to jet fuel as measured by a punching shear test is shown in Fig. 1. Here it may be seen that a void content less than 4% based on saturated surface dry specific gravity of the aggregate is required for high strength retention.

Rolling studies at field installations, using steel rollers weighing from 3.5 tons to 12 tons, have shown that the desired density and low void content can be obtained by making the break-down roll with 10 ton or 12 ton steel rollers followed by surface finishing with a rubber-tired roller with a weight of 2,000 lb per wheel and 90 psi tire pressure.

PROPERTIES AND PERFORMANCE OF EPOXY ASPHALT CONCRETE

To determine the potentialities of this new paving material, a program has been undertaken in which mechanical properties have been investigated in the laboratory, and performance under conditions of actual use has been determined in the field. The field installations, consisting of $\frac{1}{2}$ in. to 1 in. thick overlay pavements, have been placed and studies have been made with mixes using dense-graded aggregates with a maximum particle size of $\frac{1}{4}$ in.

Stability and Bearing Capacity.—As mentioned earlier, a chemical reaction takes place in the binder which converts it from a viscous liquid to a nonmelting plastic. This reaction is still in progress when the hot mix is spread and compacted, and continues for some time in the pavement. A convenient method of following the progress of the reaction in the pavement is to determine the bearing capacity of the pavement periodically. The 90° cone penetrometer test commonly used in soil test work has been shown[7] by L. W. Nijboer to give bearing capacity results in good agreement with the theory of L. Prandtl[8] when used on sand sheet mixes of the type involved in epoxy asphalt overlays. The bearing capacity measured is equivalent to the tire pressure which can be tolerated. Fig. 2 shows data taken on an EAC overlay placed at an aircraft maintenance base in the San Francisco, Calif., area. This overlay was able to support a truck with a 17,700 lb rear axle load without shoving within 15 min after compaction with steel rollers. From Fig. 2 it can be seen that 1 day after placement, the pavement could withstand a tire pressure of 200 psi and this increased to 740 psi within 1 week. A bearing capacity of 1,000 psi was reached within 10 days and values in the range 2,000 to 3,000 were attained in 30 days. Air temperatures during this period were in the range 55°F to 75°F and pavement temperatures did not exceed 105°F. At higher ambient temperatures a more rapid rise in bearing capacity is obtained.

By means of the Marshall test the stability of epoxy asphalt concrete has been determined at several levels of binder content. For comparison, asphaltic concrete specimens were prepared from the same crushed aggregate with 85/100 penetration grade asphalt as the binder. The specimens were prepared

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[7] "Plasticity as a Factor in the Design of Dense Bituminous Pavements," by L.W. Nijboer, Elsevier Publishing Co., Inc., New York, p. 109.

[8] "Concerning the Hardness of Plastic Bodies," by L. Prandtl, Nachrichten von der Königlichen Gesellschaft der Wissenschaften zu Göttingen, Math. Phys., 1920, p. 74.

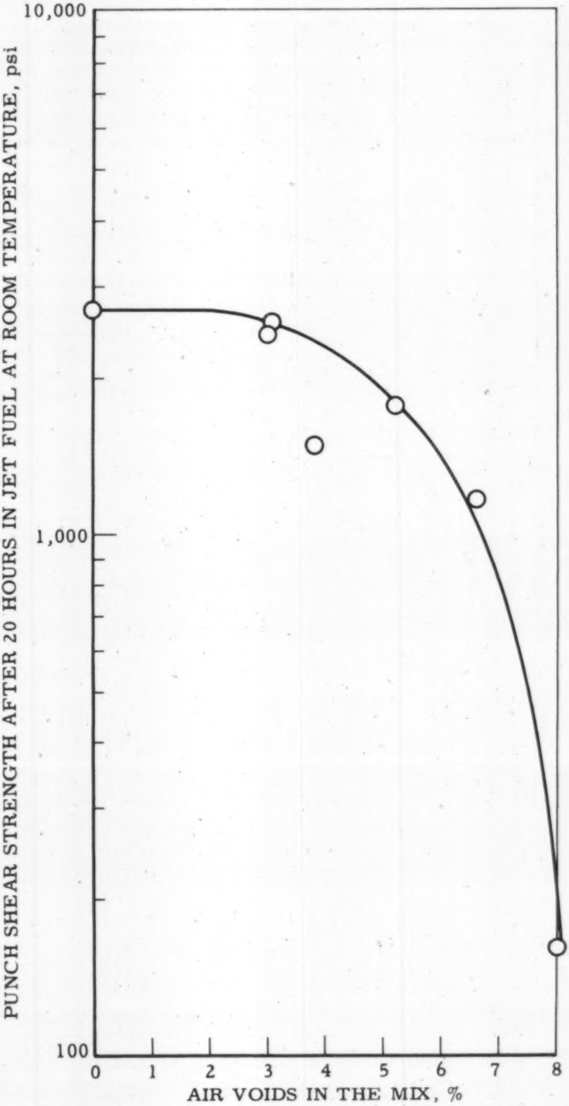

FIG. 1.—INFLUENCE OF AIR VOID CONTENT ON PUNCH SHEAR STRENGTH AFTER 20 HR OF IMMERSION IN JET FUEL AT ROOM TEMPERATURE

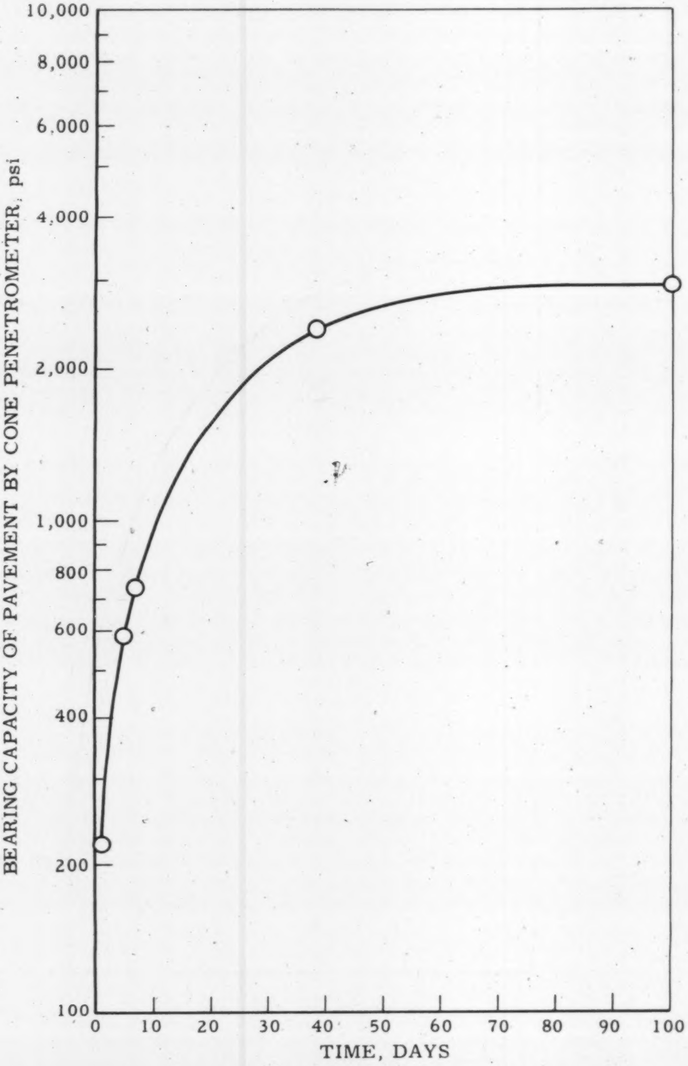

FIG. 2.—INCREASE OF BEARING CAPACITY WITH TIME

with 75 blows on each face and were cured 4 hr at 250°F. The results given in Fig. 3 show the stability of EAC to be in the range of 15,000 lb to 20,000 lb as compared with a range of 800 lb to 3,500 lb for conventional asphaltic concrete made from this dense-graded crushed aggregate. There is little apparent damage to the EAC specimens during Marshall testing, and when the load is removed the specimen rebounds, about 60% to 70% of the "flow value" actually being an elastic and recoverable deformation as shown in Table 2. Asphaltic concrete does not show this type of recovery but remains permanently deformed after the Marshall test.

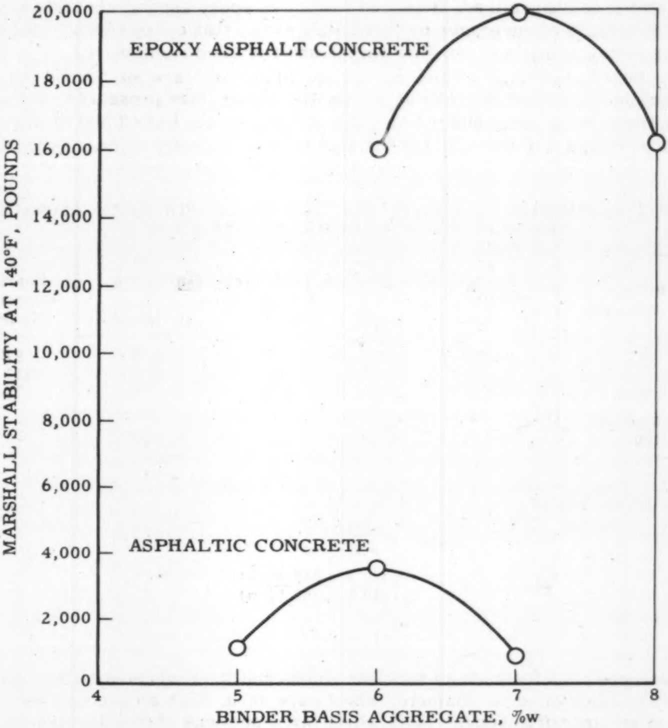

FIG. 3.—MARSHALL STABILITY OF EPOXY ASPHALT CONCRETE AND ASPHALTIC CONCRETE

A recent analysis of the Marshall test by C. T. Metcalf[9] has led to the development of a procedure for calculating the bearing capacity in terms of tire pressure which the pavement can tolerate at 140°F without the production of a compressive strain exceeding 1%. The values of bearing capacity calculated by this new procedure correlated well with field experience in 84 out of 91 pavements studied by Metcalf. In Table 2 the bearing capacities of epoxy asphalt comcrete are shown to be in the range of 700 psi to 1,100 psi as compared with 250 psi for the asphaltic concrete made with the same aggregate. This represents an improvement which is of particular significance in view of the high tire pressures and wheel loadings which are encountered in modern military aircraft.

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[9] "Use of the Marshall Stability Test in Asphalt Paving Mix Design," by C.T. Metcalf, Highway Research Bd. Bulletin, No. 234.

The ability of pavements to retain high load-carrying capacity after many repeated loadings is particularly important in taxiways and runways where aircraft traffic is channelized. Repeated loading of epoxy asphalt concrete simulating thousands of coverages by traffic has shown that epoxy asphalt concrete retains high stability and low flow values and resists densification.

The bearing capacity values mentioned previously are far beyond the requirements of aircraft now (1960) in use, the highest tire pressures currently encountered being about 500 psi on some aircraft of the United States Navy. A practical example of the need for the high bearing capacity of EAC is found in areas where vehicles such as fork-lift trucks and steel-wheeled dollies carrying heavy loads on small diameter wheels are used. Such a case was encountered in an aircraft assembly plant where large sections of structural members were being transported on trains of steel-wheeled dollies with wheel loads of 800 lb per in. of wheel width. The rough, cracked surface of the portland cement concrete floor was causing damage to the aircraft sections and this condition was alleviated by placement of a $\frac{1}{2}$ in. EAC overlay to serve as a new smooth-running surface. The overlay was placed during a weekend and had developed sufficient bearing capacity by the following Monday to support the heavy loads without indentation of the pavement.

TABLE 2.—MARSHALL STABILITY AND FLOW VALUES FOR EPOXY ASPHALT CONCRETE AND ASPHALTIC CONCRETE AT 140°F

% Binder Basis Aggregate	Average Marshall Test Values from Duplicates				Bearing Capacity,[a] in psi
	Stability, in pounds	Flow, in Inches			
		Total	Rebound	Permanent	
6	15,900	0.17	0.12	0.05	973
7	19,900	0.18	0.12	0.06	1135
8	16,100	0.22	0.13	0.09	706
6% Asphalt 85/100	3,560	0.14	0	0.14	275

[a] Tire pressure which can be tolerated at 140°F without producing a compressive strain in excess of 1%:

$$\text{Bearing Capacity} = \frac{\text{Stability}}{5 \times \text{Flow}} (K \mp 2) \quad (1-0.055\ K)$$

$$\text{where } K = \frac{1 + \sin\ (59.4-0.942\ \text{Flow})}{1 - \sin\ (59.4-0.942\ \text{Flow})}$$

Fuel and Solvent Resistance.—The resistance of epoxy asphalt concrete to deterioration by jet fuel and various solvents is an important property in many applications where the overlays are used to protect underlying pavements. To prevent fuels and solvents from seeping through the overlay it should be impermeable. This condition is achieved by the use of a dense-graded aggregate and relatively high binder content in the mix, and by the use of heavy rollers in the compaction operation. The solvent resistance of the overlay itself is also improved by high densities and low air void content as shown in Fig. 1, which shows the results obtained on cores from field installations. The resistance of $\frac{1}{2}$ in. thick specimens to the shearing action of a $\frac{7}{16}$ in. diameter steel rod driven at a rate of 0.2 in. per min by a testing machine is designated as the punch shear strength. The punch shear strength of specimens after 20 hr of immersion in jet fuel at room temperature decreases as the air void content increases. On the basis of data of this type from a number of field installations, an air void content of 4% (based on saturated surface dry specific gravity of the aggregate) has been set as a goal in the construction of overlays where solvent resistance is important. The relative solvent resistance of low air void content asphaltic concrete and epoxy asphalt concrete to jet fuel at 140°F is shown in Fig. 4. Asphaltic concrete disintegrates within 6 hr under these severe conditions while EAC retains a Marshall stability of about 15,000 lb.

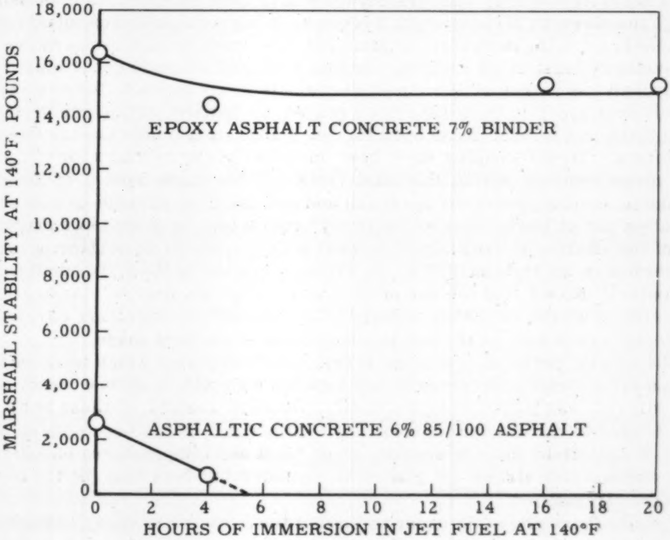

FIG. 4.—SOLVENT RESISTANCE OF EPOXY ASPHALT CONCRETE COMPARED WITH ASPHALTIC CONCRETE

The solvent resistance of epoxy asphalt concrete pavements has been tested under severe conditions in field installations. One of the early installations was in a bulk depot in which fuels and lubricants are loaded into tank trucks. A small amount of spillage and dripping occurs constantly and frequent replacement of asphaltic concrete was required. A $\frac{1}{2}$ in. overlay of EAC has been effective in protecting the underlying pavement from softening by fuels and lubricants for several years.

An extremely severe condition of solvent and fuel damage was encountered at the maintenance base of a commercial airline in the San Francisco area. As part of the maintenance and overhaul routine, planes are placed on a designated area where engines soiled by oil leaks are washed down with a petroleum solvent, resembling paint thinner, which falls on the pavement. Oil filter changes are made with some spillage; fuel tank drains at eighteen locations on the aircraft are opened to remove water and some highly aromatic aviation gasoline as well. Hydraulic fluids of both the oil base and alkyl aryl phosphate type (such as Skydrol) frequently accumulate in small amounts on the pavement. As a result of outstanding performance of an EAC overlay in this severe service, the airline elected to protect a 3-acre area of asphaltic pavement at a new jet maintenance base with an epoxy asphalt overlay.

Jet Blast Resistance.—Laboratory tests have been used to obtain an indication of the strength of epoxy asphalt concrete at high temperatures but no means has been found in the laboratory to simulate the combined high temperature and high-velocity blast of jet engines. Actual field performance under blast from jet aircraft has therefore been used to evaluate epoxy asphalt concrete. Epoxy asphalt overlays 1-in thick have been placed on spalled portland cement concrete slabs in overhaul areas used for jet planes at two military air bases in California. Thermocouples have been installed in the overlays for temperature measurements during the blast tests. These tests used a cycle which simulated normal pretakeoff operation and consisted of periods of idle power operation and of 100% power or "military" operation. In other tests prolonged use of the afterburner was also included which resulted in surface pavement temperatures as high as 800°F. A survey reported by W. J. Turnbull and C. R. Foster[10] shows that the use of afterburners occurs only on planes already in motion, with the exception of maintenance operations which are carried out in special areas such as the one in which these tests were made.

The overall performance of the overlay in these tests, which involved conditions far exceeding the severity and duration expected in normal use by military aircraft was judged to be excellent. Detailed results of these tests may be released by the military agencies at a later date. It has been pointed out[11] by J. H. Litchfield that the severity of jet blast and temperatures encountered with commercial airline jet planes is considerably less than in the case of military aircraft.

Tensile Properties.—In order to predict the behavior of epoxy asphalt concrete under various conditions of loading at various temperatures, a knowledge of its major mechanical properties is required. The tensile strength has been determined over a temperature range of 32°F to 140°F at loading times from about 1 to 10^6 sec as shown in Fig. 5 for EAC with 8% binder in the mix. The data for loading times up to about 5,000 sec were obtained on a conventional testing machine and those for longer times were produced in constant-stress creep-rupture tests. The tensile strength is rather insensitive to changes in loading time and in this respect resembles portland cement concrete much more than asphaltic concrete. Changes in tensile strength with temperature are also considerably less than those found with asphaltic concrete.

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[10] "Effects of Jet Blast and Fuel Spillage on Bituminous Pavements," by W. J. Turnbull and C. R. Foster, Proceedings, ASCE, Vol. 85, No. AT2, 1959.

[11] "Effects of Jet Fuel Spillage and Blast on Pavements," by H. J. Litchtefeld, Proceedings, ASCE, Vol. 85, No. AT3, 1959.

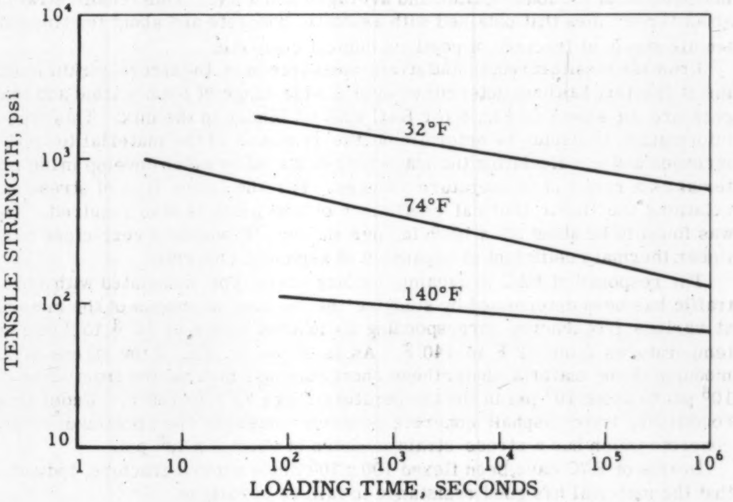

FIG. 5.—TENSILE STRENGTH OF EPOXY ASPHALT CONCRETE AS A FUNCTION OF TEMPERATURE AT LONG LOADING TIMES

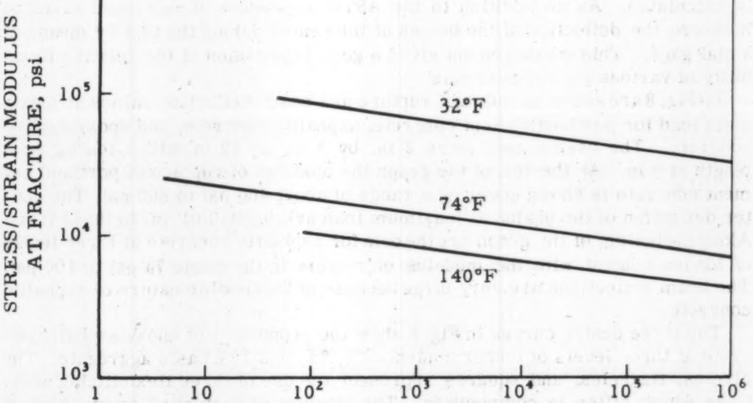

FIG. 6.—STRESS-STRAIN MODULUS OF EPOXY ASPHALT CONCRETE AS A FUNCTION OF TEMPERATURE AT LONG LOADING TIMES

The tensile strain of epoxy asphalt concrete at fracture proved to be almost independent of the loading time and averaged about 3%. This tensile strain is about three times that obtained with asphaltic concrete and about ten times the tensile strain at fracture of portland cement concrete.

From the tensile strength and strain measurements, the stress-strain modulus at fracture has been determined over a wide range of loading time and temperature as shown in Fig. 6 for EAC with 8% binder in the mix. This type of information is useful in determining the response of the material to applied stresses and in calculating the magnitude of stresses which develop in the material as a result of temperature changes. For the latter type of stress calculations the linear thermal coefficient of expansion is also required. This was found to be about 20 x 10^{-6} in. per in. per °F which is very close to the linear thermal coefficient of expansion of asphaltic concrete.

The response of EAC to dynamic loading of the type associated with moving traffic has been determined by studying the bending of beams of the material at various frequencies corresponding to loading times of 10^{-3} to 1 sec and temperatures from 32°F to 140°F. As is shown in Fig. 7 the stress-strain modulus of the material under these short loading times varies from about 3 x 10^6 psi to about 10^5 psi in the temperature range 32°F to 140°F. Under these conditions, epoxy asphalt concrete behaves somewhat like portland cement concrete which has a stress-strain modulus of about 3 x 10^6 psi.

Beams of EAC have been flexed 100 x 10^6 times without fracture, indicating that the material has good resistance to failure by fatigue.

Flexural Strength.—The flexural strength or modulus of rupture of pavements is an important property which characterizes their behavior under loading. This measurement has been made according to the procedure specified in ASTM methods C293-57T or C78-49 which consists of supporting a beam of standardized dimensions near the ends and producing bending by center loading at a specified rate with a testing machine. From the maximum load and the dimensions of the specimen, the modulus of rupture or flexural strength is calculated. As an addition to the ASTM procedure it was found useful to measure the deflection of the beams at the center during the test by means of a dial gage. This measurement gives a good impression of the relative flexibility of various paving materials.

In Fig. 8 are shown modulus of rupture and beam-deflection values at maximum load for portland cement concrete, asphaltic concrete, and epoxy asphalt concrete. The beams used were 2 in. by 3 in. by 12 in. with a testing span length of 9 in. At the left of the graph the modulus of rupture of portland cement concrete is shown covering a range of about 400 psi to 800 psi. The center deflection of the beams at maximum load averaged 0.01 in. in these tests. Along the bottom of the graph are the data for asphaltic concrete at three levels of binder content with the modulus of rupture in the range 75 psi to 100 psi. The beam deflections are very large because of the flexible nature of asphaltic concrete.

The three center curves in Fig. 8 show the properties of epoxy asphalt concrete at three levels of binder content—7%, 8%, and 12% basis aggregate. The circles, triangles, and squares represent the use of three flexibilizing additives which differ in composition. The modulus of rupture of epoxy asphalt concrete may be varied from about 400 psi, which is about the lower level for portland cement concrete, to about 2,400 psi, which is two to three times that obtained with the best portland cement concrete. At the same time, epoxy asphalt concrete may be flexed to the same extent as asphaltic concrete at maxi-

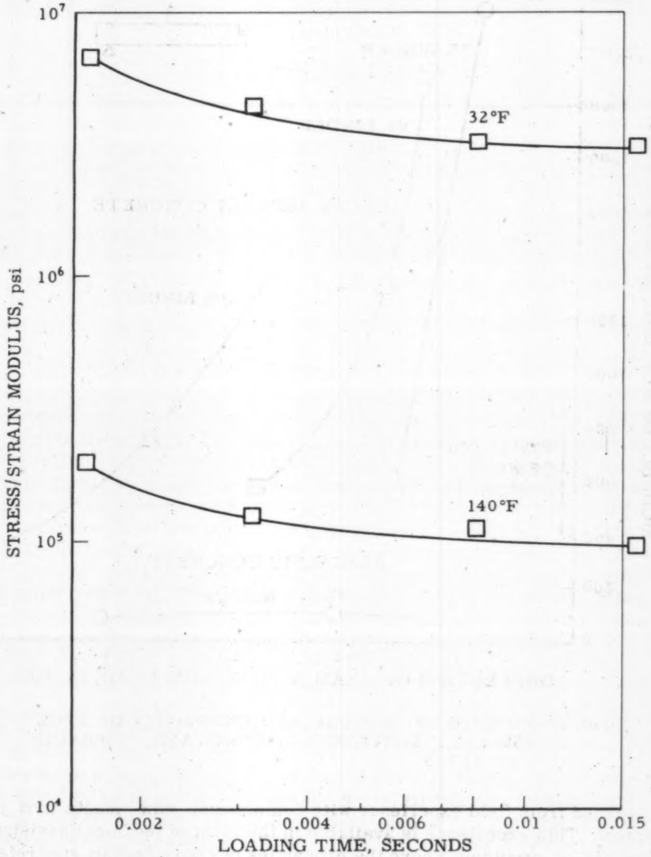

FIG. 7.—STRESS/STRAIN MODULUS OF EPOXY ASPHALT CONCRETE AT SHORT LOADING TIMES

mum load. Thus we have a structural material which combines the desirable properties of both rigid and flexible pavements: a strength equal to or greater than that of portland cement concrete and a tolerance for bending equal to that of asphaltic concrete.

AVAILABILITY OF TECHNICAL ASSISTANCE

The successful use of epoxy asphalt concrete requires proper handling of the material in the mix plant and during the laying operations. The changes from conventional hot-mix handling practices are few but important, and have

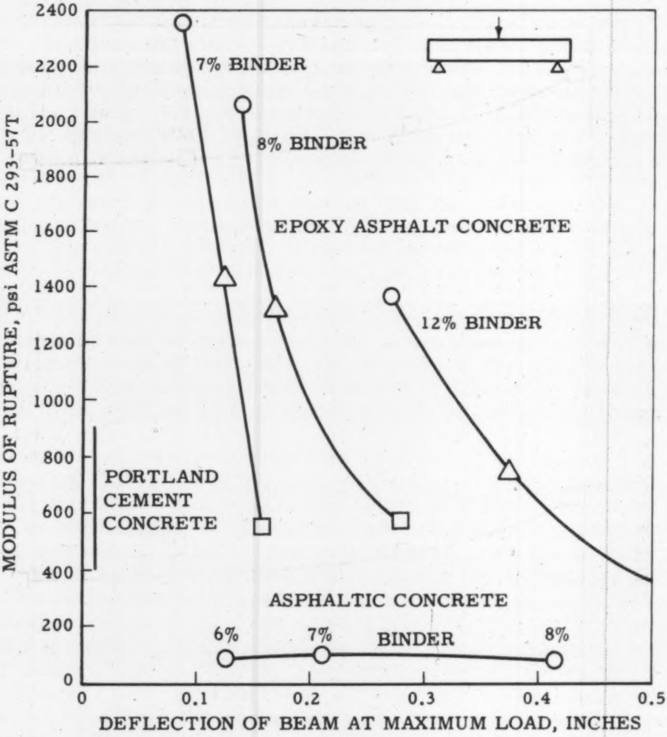

FIG. 8.—MODULUS OF RUPTURE AND FLEXIBILITY OF EPOXY ASPHALT, PORTLAND CEMENT, AND ASPHALTIC CONCRETES

been learned from field experience with commercial mix plants and paving equipment. This experience is available in the form of technical assistance in solving paving problems where the properties of epoxy asphalt concrete indicate its use to be desirable.

ACKNOWLEDGMENTS

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NAVIGATION MODEL STUDIES OF NEW OHIO RIVER LOCKS

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SYNOPSIS

The major program of modernizing navigation facilities on the Ohio River and its tributaries involves replacement of the obsolete and inadequate system of low-lift, movable-wicket dams and small locks with a smaller number of relatively high, gated dams, including greatly expanded locking facilities suitable for handling modern tows. This paper describes the successful utilization of hydraulic models to investigate navigation conditions that will be encountered at the new locks and dams. Several model studies have been and are being used to develop optimum locations and arrangements of the locks and dams and appurtenant structures, as well as methods of operation of the dam gates, that will insure the elimination of hazardous or difficult navigation conditions for tows entering or leaving the locks.

INTRODUCTION

Modernization Program.—Canalization of the Ohio River at the present time is accomplished by means of 46 locks and dams, all but two of which were completed in 1929 or earlier. 41 of these dams are of the movable-wicket type with lifts averaging only about 7 ft, and each includes one lock chamber 110 ft by 600 ft in size.

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J. W. Bruce, D. W. Keller and J. A. Neill have presented[3] descriptions of the physical deterioration and obsolescence of the existing structures and the vastly increased demands of river traffic that make the modernization and replacement of existing navigation facilities on the Ohio River system, as well as the general requirements and criteria that establish the characteristics of the replacement facilities a necessity.

In order to provide modern facilities suitable for the efficient passage of current traffic as well as that which can reasonably be anticipated, a major program of replacement and modernization has been undertaken, and the replacement structures are now in various stages of construction or planning by the various districts of the Corps of Engineers, Dept. of the Army, under the supervision of the Ohio River Division. The old movable-wicket dams, with sills at river-bed elevation to permit open-river navigation at high stages, are being replaced by a smaller number of relatively high, gated dams (Fig. 1). The new dams will create deeper and longer pools—some more than 100 miles in length—and each will have a main lock with clear dimensions of 110-ft by 1200-ft and a 110-ft by 600-ft auxiliary lock.

Design for Navigability.—River travel will be speeded by the longer pools, requiring fewer lockages, and by the longer locks that will largely eliminate the present necessity for double or triple lockages to pass individual tows of optimum length. However, the new type structures also produce certain navigation and hydraulic design problems not encountered with the old movable-wicket dams. The maximum length of tow passing through the locks will be doubled. Also, since the gated dams will not permit open-river navigation during high flows as did the wicket dams, these longer tows will have to use the locks at much higher flows when current velocities in the approaches to the locks will be several times greater than those encountered heretofore.

Tows approaching a lock can be affected by adverse currents (Fig. 2) to such an extent as to cause or contribute to accidents or delays. The currents within the approaches to the lock or locks are controlled to a large extent by the configuration and alignment of the channel upstream and downstream therefrom, and are affected by the sudden contraction, then expansion, of the channel produced by the obstruction of the locks and guard walls.

The effect of such a contraction on currents in the upper approach is to produce an outdraft or cross-current that affects the movement of a down-bound tow at a time when its rudder power is reduced because of reduced speed with respect to the river currents. The intensity of cross-currents of this type is dependent upon the position of the locks with respect to the bankline, the total width of the locks and guard wall, the velocity and alignment of the river currents upstream of the locks, the width of the river channel, and the general elevation of the adjacent overbank area. The sudden expansion of the channel at the downstream end of the lower lock guard wall causes an eddy to form in the lower approach.

Currents in the eddy that affect navigation are those toward the bank at the lower end of the eddy, the upstream currents along the bank, and the outdraft or river-ward cross-currents at the upstream end of the eddy. An up-bound tow approaching the locks with little or no rudder control because of the reduced speed and upstream currents is at the mercy of the currents, which vary in position and intensity. Other currents which might affect navigation are

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[3] "Modern Facilities for Ohio River Navigation," by James W. Bruce, Dwight W. Keller, and James A. Neill, Proceedings, ASCE, Vol. 83, No. WW2, May, 1957.

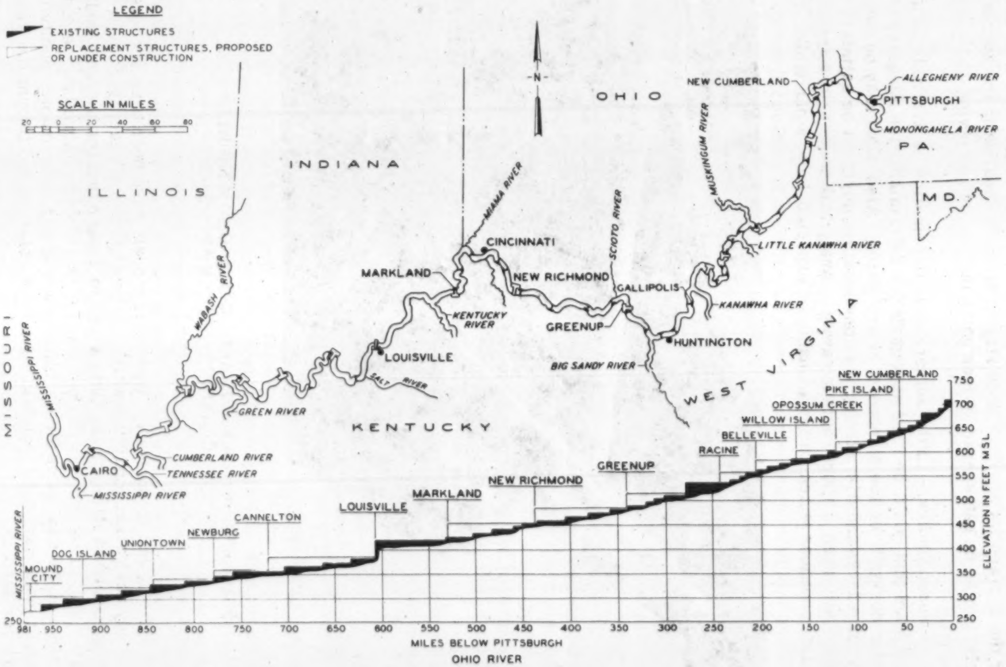

LEGEND
EXISTING STRUCTURES
REPLACEMENT STRUCTURES, PROPOSED OR UNDER CONSTRUCTION
SCALE IN MILES
20 0 20 40 60 80
-N-
ILLINOIS
INDIANA
OHIO
P.A.
M.D.
WEST VIRGINIA
KENTUCKY
MISSOURI
ALLEGHENY RIVER
PITTSBURGH
MONONGAHELA RIVER
NEW CUMBERLAND
MUSKINGUM RIVER
LITTLE KANAWHA RIVER
KANAWHA RIVER
HUNTINGTON
GALLIPOLIS
SCIOTO RIVER
BIG SANDY RIVER
GREENUP
NEW RICHMOND
CINCINNATI
MIAMA RIVER
MARKLAND
KENTUCKY RIVER
LOUISVILLE
SALT RIVER
GREEN RIVER
WABASH RIVER
CUMBERLAND RIVER
TENNESSEE RIVER
CAIRO
MISSISSIPPI RIVER
MISSISSIPPI RIVER
MISSISSIPPI RIVER
MOUND CITY
DOG ISLAND
UNIONTOWN
NEWBURG
CANNELTON
LOUISVILLE
MARKLAND
NEW RICHMOND
GREENUP
RACINE
BELLEVILLE
WILLOW ISLAND
OPOSSUM CREEK
PIKE ISLAND
NEW CUMBERLAND
ELEVATION IN FEET MSL
250 300 350 400 450 500 550 600 650 700 750
981 950 900 850 800 750 700 650 600 550 500 450 400 350 300 250 200 150 100 50 0
MILES BELOW PITTSBURGH
OHIO RIVER

FIG. 1.—PLAN FOR REPLACEMENT OR MODERNIZATION OF NAVIAGTION STRUCTURES

those resulting from lock filling and emptying, operation of hydroplants, flow from tributary streams, and so forth. Some of the other problems in hydraulics with which a design must contend are accumulation of debris and ice, swell head at the dam and its effect on flood stages, erosion, shoaling in the approach channels, forces on gates, head on lock gates during lock emptying and filling, surges, etc.

Use of Model Studies.—Little authoritative information is available in assembled form that could be used as a guide for determining the adequacy of a particular design from the standpoint of navigation. Also, what may be satisfactory for a locks and dam structure at one location might not be satisfactory at another site even a short distance away. Analytical studies to determine hydraulic conditions which can reasonably be expected from a particular design are difficult and inconclusive. Hydraulic models have been used successfully to study navigation conditions which can be expected at several of the proposed new locks and dams on the Ohio River to insure that the designs would provide the best arrangements of the locks and guard walls as well as methods of operation of the dam gates for the elimination of any undesirable conditions which might make navigation conditions for tows entering and leaving the locks difficult or hazardous.

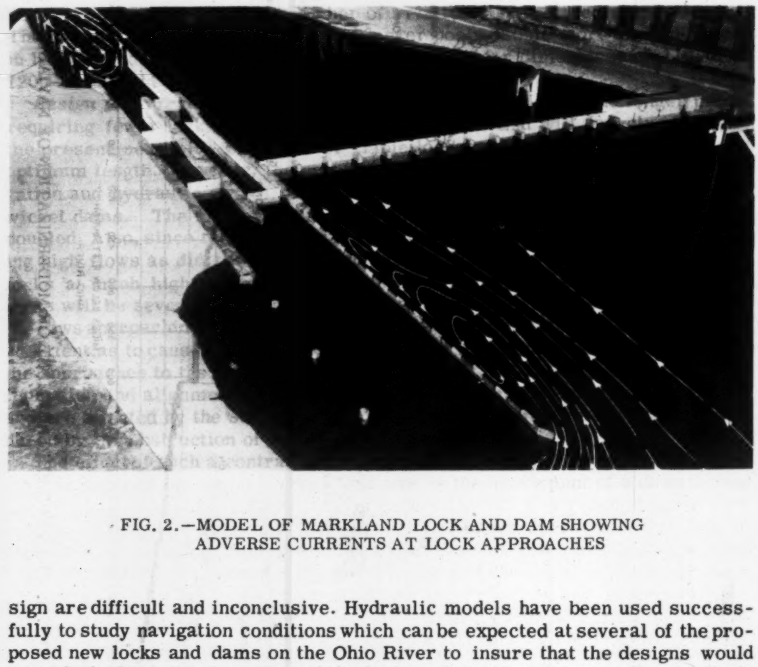


FIG. 2.—MODEL OF MARKLAND LOCK AND DAM SHOWING ADVERSE CURRENTS AT LOCK APPROACHES

Since 1952 the Corps' Waterways Experiment Station at Vicksburg, Miss., has conducted several model studies of proposed modern locks and dam structures for the Ohio River and tributaries at the request of the Louisville, Huntington, and Pittsburgh districts of the Corps of Engineers. Studies have been completed on Greenup, Markland, New Richmond, and Louisville Locks and

Dams (formerly known as Locks and Dam No. 41). Studies are underway on the Pike Island, Maxwell Locks and Dam, and Locks and Dam No. 4; the latter two structures are on the Monongahela River, a tributary of the Ohio. All of the structures are to be new except for the Louisville Locks and Dam, and Locks and Dam 4 which will require considerable revision.

THE MODELS

Description.—The models used for study on navigation problems of these projects are of the fixed-bed type illustrated in Fig. 3. The models are built to an undistorted linear scale of 1 to 120, model to prototype, to insure the accurate reproduction of velocities, cross-currents, and eddies that would affect navigation. The reach of the river reproduced in any of these models is that required to reproduce accurately the currents within the upstream and downstream approaches to the locks. The reach varies from about 3 miles to about 6 miles, depending upon the alignment of the channel in the vicinity of the location of the proposed structure, and whether the location is fixed or alternate sites are considered.

The locks and dam structures are fabricated in the model shop of wood specially treated to prevent expansion, or sheet metal, or of a combination of wood and sheet metal. Since structural details of the dam gates play no part in the navigation studies, except in controlling flow over the dam and maintaining the upper pool elevation, they are simulated with sheet metal slide-type gates for economy. Similar type gates are also used to simulate the lock miter gates. In the design of models of this type, provisions are made for the installation of alternate plans being considered and for probable changes in the design and arrangement of the lock walls, in the composition of dam, and in dredging insofar as possible. Structural models used to study details of the locks and dam structures such as the effectiveness of the stilling basin, stability of gates, etc., are constructed to scales of 1 to 25 or larger.

Model Appurtenances.—Operation of the navigation models requires special appurtenances in addition to the usual equipment for providing and controlling water supply, water-surface elevations, tailwater, and so forth. One of the most important and most used special apparatuses in the remote-controlled self-propelled towboat and tow. The towboat shown in Fig. 4 is constructed to scale, reproducing the hull of a modern-type towboat including twin screws, rudders, and Kort nozzles. The towboat is powered by batteries placed within the tow, and is operated by remote control utilizing a 4-channel radio transmitter and receiver system developed by the modification of commercially available equipment.

The towboat can be operated in forward and reverse at one-fourth, one-half and full speed. An indicator is provided to show the set of the rudder at all times and various colored lights are provided to indicate direction and speed of the propellers. A rheostat and an ammeter are also provided to permit the adjustment of the maximum speed of the towboat to represent towboats of various push powers. The towboat is calibrated under various loads, and a curve indicating the speed in miles per hour in a given depth of water is plotted against meter readings for each load. The towboat and tow are used in the models to determine the effects of currents on a tow and the difficulties encountered in approaching and leaving the locks and in negotiating bends or special reaches of the river. Among other equipment used are floats submerged

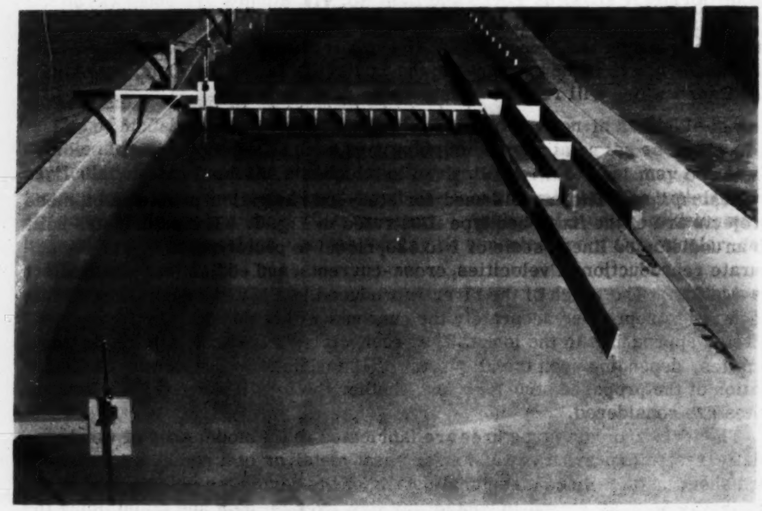


FIG. 3.—MODEL OF GREENUP LOCKS AND DAM—TYPICAL OF MODELS USED TO STUDY OHIO RIVER NAVIGATION STRUCTURES

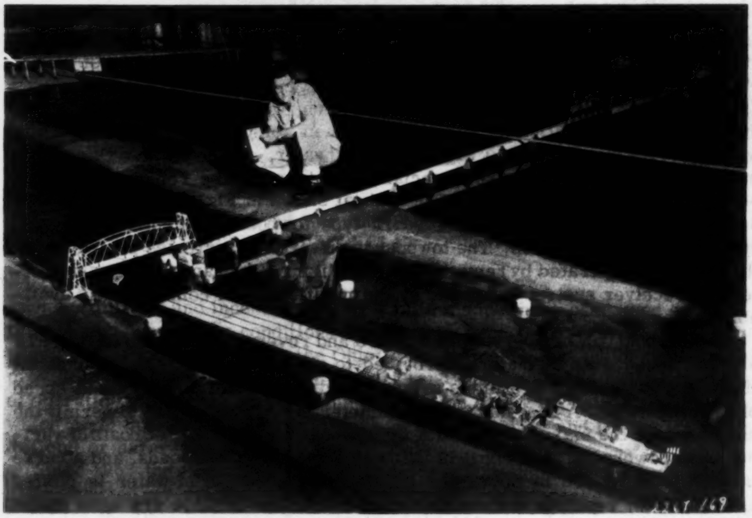


FIG. 4.—MODEL TOWBOAT AND TOW USED TO DETERMINE NAVIGATION CONDITIONS IN LOCK APPROACHES

to various depths, miniature current meters, movie and still cameras, special lighting equipment for obtaining progressive multiple exposures to show tow movements, force and displacement gages, Brush recorders, and universal amplifiers.

Testing Procedure.—The first step in the testing procedure of a navigation model of this type is to reproduce a series of selected representative river flows with the river in its natural state, that is, prior to construction of the proposed structure to be tested. Comparison of the results thus obtained with corresponding data obtained in the prototype insures that the model is accurately reproducing flow lines, current directions, and velocities of the prototype. The proposed locks and dam structure is then installed in the model and tested.

Testing of the structure involves the reproduction of several representative flows, including controlled and uncontrolled river flows. Studies are made of directions and strengths of significant currents in both approaches to the locks by plotting the paths and timing the travels of floats submerged to the depth of a loaded barge, and by measuring spot velocities in critical areas with a miniature current meter. Also, the effects of currents on tows are studied by observing the behavior of the model tow while drifting into the upper lock approach, while approaching and leaving the locks under power, and while navigating restricted areas.

Fig. 5 illustrates a photographic method of recording the maneuvering of a tow through a restricted reach in the upper approach to the Louisville Locks and Dam. Tests are also conducted on the navigation models to determine the effects of the proposed structures on high flood stages by measuring water-surface profiles before and after installation of the structures. Different structures also require investigation of special features such as effects of powerhouse operation, surges, and tributary inflow on navigation conditions.

Greenup Locks and Dam.—The results of a model study of one project have been useful in the design of other structures. The first of the series of studies of modern locks and dams for the Ohio River was that of the Greenup project. Two alternate plans for this structure (Fig. 6) were first considered, one with the main lock and long guide wall on the landside and one with the main lock and a long guard wall on the riverside. The study readily demonstrated that the latter alternate was preferable, and this arrangement has been incorporated in subsequent designs. With the main lock and long guide wall on the landside (Fig. 7), it was found that tows would have to approach the guide wall sufficiently close to attach mooring lines, and a slight misjudgment of currents by the pilot would place the tow in danger of hitting the end of the wall or of being moved too far river-ward to attach mooring lines. Tows attempting to avoid these hazards by driving ahead at an angle, toward the guide wall, to attach mooring lines, would be in danger of having their sterns moved river-ward by the cross-currents and breaking up against the end of the guard wall.

With the main lock on the landside, the eddy in the lower approach would extend upstream to the lower end of the guide wall, and the river-ward currents of the eddy at that point would move the head of an upbound tow away from the wall. Here again, a misjudgment of the currents in the eddy could cause the head of the tow to strike the end of the guide wall, or else to pass too far from the wall to permit the placing of mooring lines to snub the tow into alignment. The pilot could pass the end of the guide wall at a safe distance, drive the head of the two into the wall to attach lines, and then work the stern of the two into alignment with the wall. However, considerable time could be lost in making this maneuver against the currents.

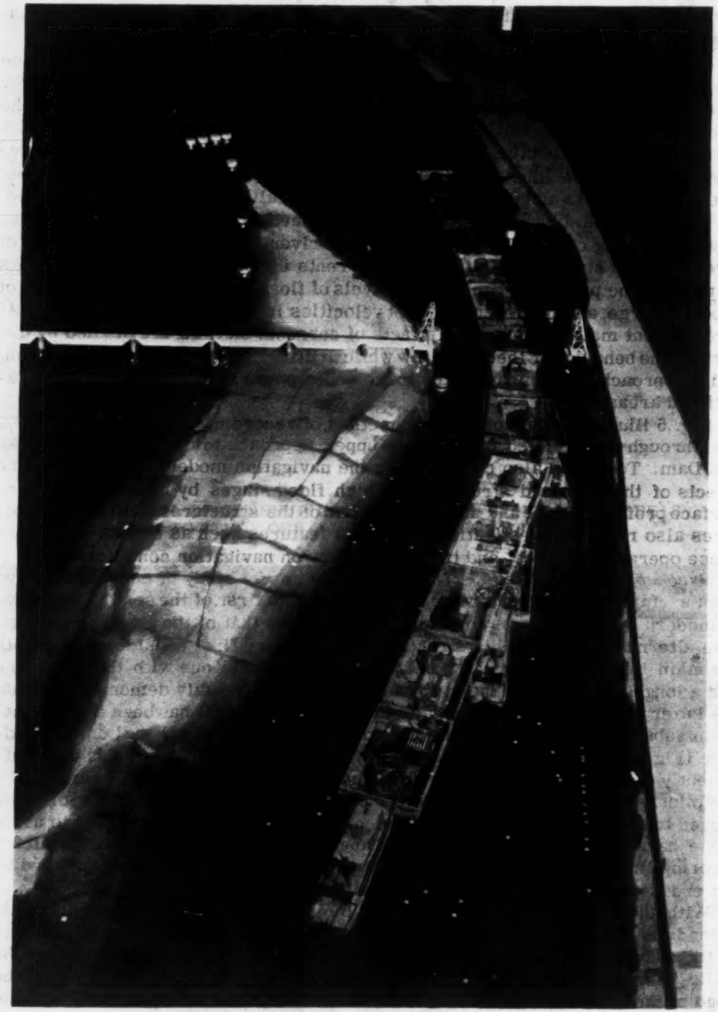
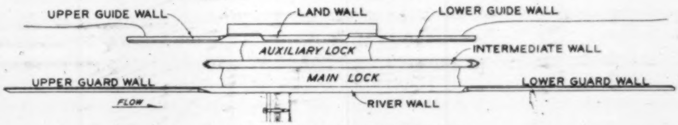


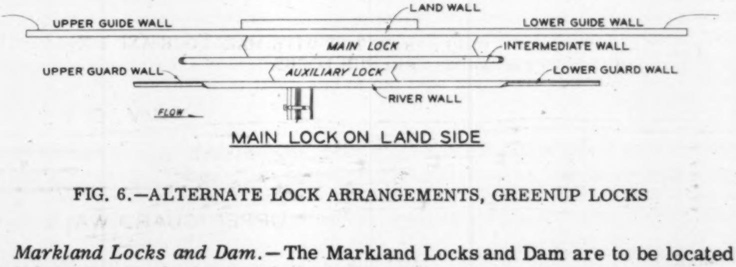
FIG. 5.—A SERIES OF INSTANTANEOUS EXPOSURES IS USED TO RECORD PHOTOGRAPHICALLY TOW MANEUVERS IN CRITICAL REACHES

With the main lock and a long guard wall located on the riverward side of the auxiliary lock, tows would enter relatively slack water before beginning to line up to enter the locks from either upstream or downstream. Even under this condition, however, the model tests indicated that ports through the upper guardwall would be required to reduce the hazardous riverward crosscurrents in the upper approach immediately above the end of this wall. Fig. 8 shows navigation conditions in the upper approach with and without ports through the guard wall, and the manner in which these conditions affect a downbound tow.

The Greenup Locks and Dam model study also demonstrated that the locks and walls could be moved riverward 100 ft to reduce the volume and cost of required dredging. In addition, the study demonstrated that the length of the intermediate wall could be reduced, and that discharge from the locks emptying system would not affect navigation in the lower approach during low flows with the main lock located on the riverside, even without special stilling arrangements.


UPPER GUIDE WALL
LAND WALL
LOWER GUIDE WALL
AUXILIARY LOCK
INTERMEDIATE WALL
UPPER GUARD WALL
MAIN LOCK
LOWER GUARD WALL
FLOW
RIVER WALL

MAIN LOCK ON RIVER SIDE


LAND WALL
UPPER GUIDE WALL
LOWER GUIDE WALL
MAIN LOCK
INTERMEDIATE WALL
UPPER GUARD WALL
AUXILIARY LOCK
LOWER GUARD WALL
RIVER WALL
FLOW

MAIN LOCK ON LAND SIDE

FIG. 6.—ALTERNATE LOCK ARRANGEMENTS, GREENUP LOCKS

Markland Locks and Dam.—The Markland Locks and Dam are to be located just downstream of a bend with the locks on the outside of the curve. Because of this position, flow will be concentrated on the lock side of the river and velocities in the approaches will tend to be higher than those encountered at the Greenup Locks. The results of tests of this structure indicated that ports would be required in the upper guard wall to reduce the intensity of the crosscurrents and that removal of a prominence and other irregularities along the left bank, above the guide wall, would be required to improve the alignment of currents in that vicinity and facilitate the proper aligning of tows prior to their entering the upper lock approach.

The study also indicated that dredging of the right bank opposite the lock would reduce the concentration of flow along the left bank above the guard wall, thus reducing velocities and improving the alignment of currents in the approach. Development on the model of a modified alignment for the dredge cut resulted in a reduction in the amount of dredging without affecting the results. The model also indicated that with this dredging the number of dam gates could be reduced from 13 to 12 without affecting navigation conditions.

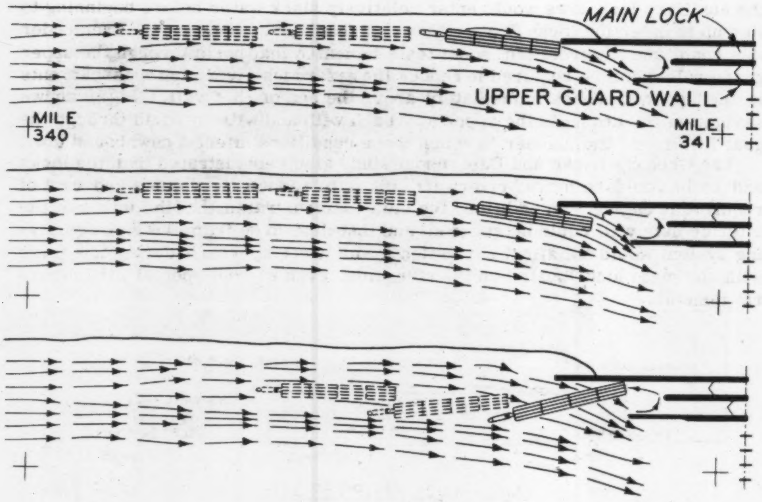

MAIN LOCK
UPPER GUARD WALL
MILE 340
MILE 341

FIG. 7.—EFFECTS OF CURRENTS ON TOW WITH MAIN LOCK AND LONG GUIDE WALL ON LANDSIDE, GREENUP LOCKS

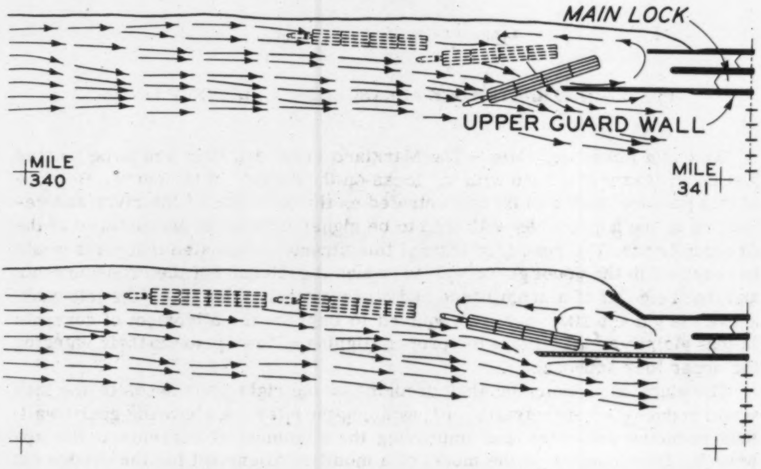

MAIN LOCK
UPPER GUARD WALL
MILE 340
MILE 341

FIG. 8.—EFFECTS OF PORTS IN UPPER GUARD WALL, GREENUP LOCKS

Tests were also conducted to determine the effectiveness of an ice and debris chute in the upper guard wall and the effects of powerhouse operation and the Steven Creek diversion canal discharges on navigation conditions. The results of these tests indicated that: 1) the debris chute would be effective in clearing the upper approach of ice and drift but would produce conditions hazardous to navigation; 2) powerhouse operation during low river flow would produce a large eddy or "backlash currents" which would extend into the lower lock approach but that the eddy could be broken up by the partial opening of the third gate from the lock; and 3) the length of the Steven Creek diversion canal could be reduced appreciably without affecting navigation.

New Richmond Locks and Dam.—The New Richmond Locks and Dam structure is proposed for construction in a relatively straight reach of the river. The original design included ports in the upper guard wall and no major changes were indicated. The model study indicated that the top of the ports in the upper guard wall should be lowered to an elevation of 15 ft below normal pool to reduce the tendency for tows to be pinned against the guard wall and to facilitate the movement of tows approaching the auxiliary locks.

Because of the elevation of the overbank adjacent to the locks, considerable overbank flow occurs during the maximum navigable stage. Placing spoil along the overbank so as to reduce overbank flow bypassing the locks and dam would increase the amount of flow the guard wall tends to intercept and thus the intensity of the outdraft at the end of the wall. Moving the locks and lock walls landward 50 ft would tend to reduce the intensity of these crosscurrents at the end of the wall.

Louisville Locks and Dam.—The Louisville Locks and Dam (formerly Locks and Dam No. 41) is an existing structure requiring considerable revision. The dam consists of two bear traps each 91 ft long just upstream of a hydroelectric plant, and the remainder of fixed and of old type maneuverable dam for a total of 8700 ft.

The dam, placed in operation in 1927, is in constant need of maintenance and is dangerous to operate. The locks placed in operation in 1930 have a 37-ft lift and consist of one 110-ft by 600-ft lock and one 56-ft by 360-ft lock. Entrance to the locks from the upstream side is through a canal approximately 2 miles long and 200 ft wide throughout most of its length. Because of this long narrow approach channel, high lift, and slow lock filling and emptying characteristics, the Louisville Locks and Dam is one of the most critical traffic bottlenecks on the Ohio River.

The Louisville Locks and Dam model reproduced about 6 miles of the Ohio River with the existing locks and dam structures, approach canal, hydroplant, bridges, levees, walls, buildings, and such other structures as affect flow conditions in the prototype. Construction of the existing structures was required to provide a basis for adjustment and verification of the model with measured prototype conditions. The model reproduced satisfactorily the measured water-surface elevations, velocity and current direction measurements, and surges in the approach canal resulting from lock filling.

The proposed improvement of conditions at the Louisville Locks and Dam involved the construction of a new 1200-ft by 110-ft lock, elimination of the restricted approach channel, and revision of the existing dam. A large number of alternate plans and modifications were tested before a satisfactory solution was reached.

The design of the approach channel was dictated to a large extent by surge conditions that could affect navigation conditions within the channel and operation of the lock gates. The existing 600-ft lock is filled in about 18 min. This filling operation produces a surge varying from about 1 ft (low to high) to about 3 ft depending upon the number and timing of lock filling operations. The 1200-ft lock will be filled within about 9 min which will increase the rate of filling and tendency for surge about 4 times. In this case the model was used to study surges at critical points within the approach channel and at the powerhouse.

Also, in connection with the approach channel, the placement of a large quantity of dredge spoil and its effect on navigation conditions and on flood stages were investigated. As far as the dam was concerned, several plans including gated structures were tested and rejected on the basis of model indications of their effects on flood stages, high velocities in critical areas, and, in one instance, the indication that six of the proposed 14 gates would pass little or no flow during flood stages. In the lower approach, tests were conducted to eliminate undesirable flow conditions and to reduce the head on the lower lock gate at the end of lock emptying operations.

The model study of the Louisville Locks and Dam resulted in the development of satisfactory plans which included a 500-ft approach channel with a surge basin, the moving of one lock-filling intake so as to draw water from the surge basin, a ported lower lock guard wall with dredging along sand Island, the establishing of the grade and location of spoil, and revisions to the dam.

Other studies are in progress (as of October, 1958). The study of Pike Island Locks and Dam is one of these. Several plans and modifications have been tested including changes in the locations and alignment of the locks and dam and the position of the locks with respect to the dam. The results of these tests and tests of the two Monongahela River Locks and Dam have not progressed sufficiently to permit the drawing of definite conclusions.

CONCLUSIONS

The value of model investigations of navigation projects on the Ohio River has been well demonstrated. As a result of these investigations certain design features have been more or less standardized. Also, considerable general knowledge has been gained on certain other design features, methods of evaluating navigation conditions, and the type of structures desired by or acceptable to navigation interests.

Placing the main lock on the riverside of the auxiliary lock and including ports in the upper guard wall where locks and dams are located in the open river channel, have become accepted as a general requirement. Ports in the upper guard wall not only improve navigation conditions but also tend to reduce scour at the end of the wall. They further tend to minimize the swell-head produced by the structures through reduction in the size of the eddy on the riverside of the wall, which improves the distribution of flow over the dam.

The number and size of ports required in the upper guard wall vary somewhat with each condition, depending to a large extent upon the amount of river flow the wall tends to intercept and the hydraulic efficiency of the ports used. Increasing the hydraulic capacity of the ports will increase the ease with which a tow can be made to drift into the upper lock approach. On the other hand, increasing the capacity of ports will also increase 1) the tendency for ice and drift to be trapped in the upper lock approach, 2) the difficulty of tows approaching the guide wall for entrance into the auxiliary lock, 3) the tendency for tows to be pinned against the guard wall, and 4) the danger of tows hitting the upper lock gate because of the increase in the speed of approach while drifting. Lowering the top elevation of the ports will tend to reduce the tendency for tows to be pinned against the guard wall and the difficulty of tows entering the auxiliary lock.

Because of the nature of stream flow, the configuration and alignment of a river channel is never exactly the same in any two reaches. For this reason, design features found satisfactory for a lock and dam structure in one locality cannot always be applied to another location. Observation of a model of a project in operation often brings to light certain undesirable design features or certain problems that will require special treatment which cannot be determined from an analysis of prototype conditions or which may not have been recognized as a design consideration. Each of the model studies of Ohio River locks and dams conducted at the Waterways Experiment Station has resulted in major changes or modifications in the original design except in the case of the New Richmond Locks and Dam where only minor revisions were indicated as desirable.

The amount of savings in initial and future costs attributable to these model studies is difficult to evaluate. The studies have resulted in a decrease in the number of dam gates required, in decreasing dredging volumes, and in the elimination of certain structures found to be ineffective or not required. Of still greater significance have been the design modifications that will greatly improve the navigability of the ultimate structures in the river through the elimination of navigation hazards, scme of which would probably have otherwise required modification of the structures after their construction in the river.

The models have also proven to be a valuable tool for demonstrating the functioning of the proposed locks and dams to navigation interests who will be vitally concerned with navigating the Ohio and its tributaries. These demonstrations, arranged by the Ohio River Division and the Districts concerned, have produced two substantial benefits: 1) experienced navigation people have contributed valuable suggestions for improving the navigability of the proposed structures; and 2) navigation interests have been satisfied in advance as to the adequacy of the structures to meet their needs.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3162

CANADIAN SECTION OF THE ST. LAWRENCE SEAWAY

By Lawrence H. Burpee,[1] M. ASCE

SYNOPSIS

This paper describes the St. Lawrence Seaway project and summarizes many of the facts relating to the navigation features of the Canadian share of the work. Preliminary facts relating to operation are given.

INTRODUCTION

The Canadian share of the St. Lawrence Seaway project comprises all of the navigation work in the Canadian section of the St. Lawrence River which extends for a distance of 77 miles from Mile 0 at the entrance from the harbor of Montreal to the head of Lake St. Francis, together with several features of the project in the 113-mile International Section of the River upstream from Lake St. Francis to Lake Ontario.

During the past five years (1955-1960), the combined power and navigation development of the River has been completed. Power in the International Section has been developed by The Power Authority of the State of New York and by The Hydro-Electric Power Commission of Ontario, and power in the Canadian section has been developed by the Quebec Hydro-Electric Commission. Navigation works have been constructed by the Corps of Engineers for The Saint Lawrence Seaway Development Corporation (United States), and by The St. Lawrence Seaway Authority (Canada). The comprehensive development of the

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St. Lawrence River has been a great international project, in which five constructing entities have taken part. The cooperation has been excellent.

Regarding the construction of the navigation works, it was agreed between the United States and Canada in August 1954 that the entire Seaway would be completed and ready for operation by the opening of the 1959 season. Decisions were also made in August 1954 which set the standards to which all of the navigation works would be built.

This paper deals with the parts of the Seaway constructed by Canada.

Fig. 1 shows the St. Lawrence and the Great Lakes Region. Fig. 2 shows the river from Montreal Harbor to Lake Ontario, and on it are shown the areas covered by the work. Fig. 3 is an enlargement of the 48-mile reach of river from Montreal Harbor up to the foot of Lake St. Francis, in which there is a rise of 133 ft, made up of 50 ft in the Lachine Rapids and 83 ft in the Soulanges Rapids. Four of the five Canadian locks are in this reach. The mileages shown on Fig. 3 will serve to locate the various locks and other structures.

The work constructed in the Canadian section of the River includes an $18\frac{1}{2}$-mile side canal and two locks to bypass the Lachine Rapids, modifications to the 15-mile Beauharnois Power Canal (Quebec Hydro) and two locks to bypass the Soulanges Rapids, dredging, modifications to or construction of eleven bridges, construction of a four-lane highway tunnel under Lower Beauharnois Lock, and miscellaneous railway, highway, power and municipal relocations.

The work constructed by Canada in the International Section of the River includes the short canal and the lock to bypass the Iroquois Control Dam, part of the dredging, some channel improvements, and a share in the Cornwall-Massena International Bridge.

The cost of the Canadian section of the navigation works, split up into the several areas, is presented in Table 1.

Water Levels.—The water level at the Seaway entrance in Montreal Harbor has varied during the open-water season from an extreme low of 15 ft above mean sea level to a flood stage of 33 ft. The range of water levels on Lake St. Louis (Mile 19-31) has been from 65 ft to 75 ft above mean sea level, with a normal elevation of around 69 ft. The water levels at Montreal and on Lake St. Louis are influenced to a large extent by the wide range of flow of the Ottawa River.

Lake St. Francis (Mile 47-77) has a natural range of level between 149 ft and 154 ft. In 1935, the series of control dams across the four channels at the outlet of Lake St. Francis were completed as features of the Beauharnois Power Development, and the water level of the Lake has, for many years, been generally controlled at a minimum elevation of 152 ft.

The new Lake St. Lawrence, (Mile 87-112) created by the St. Lawrence Power Project, will have a water elevation ranging from 238 ft to 242 ft. The slope in the upstream end of Lake St. Lawrence up to the Control Dam at Iroquois, and from the dam to Lake Ontario, will vary according to the plan for regulation of Lake Ontario levels and outflows established by International Agreements. Lake Ontario will have a controlled range of water level generally between 244 ft and 248 ft.

Channels.—All channels have a minimum depth of 27 ft. The channel below the St. Lambert lock (Mile 3) to Montreal Harbor has a bottom width of 225 ft, which was considered to be the minimum for good operation where ships first enter the Seaway. The bottom width of the channel above St. Lambert Lock and around Laprairie Basin to Côte Ste-Catherine Lock (Mile 12) was adopted at

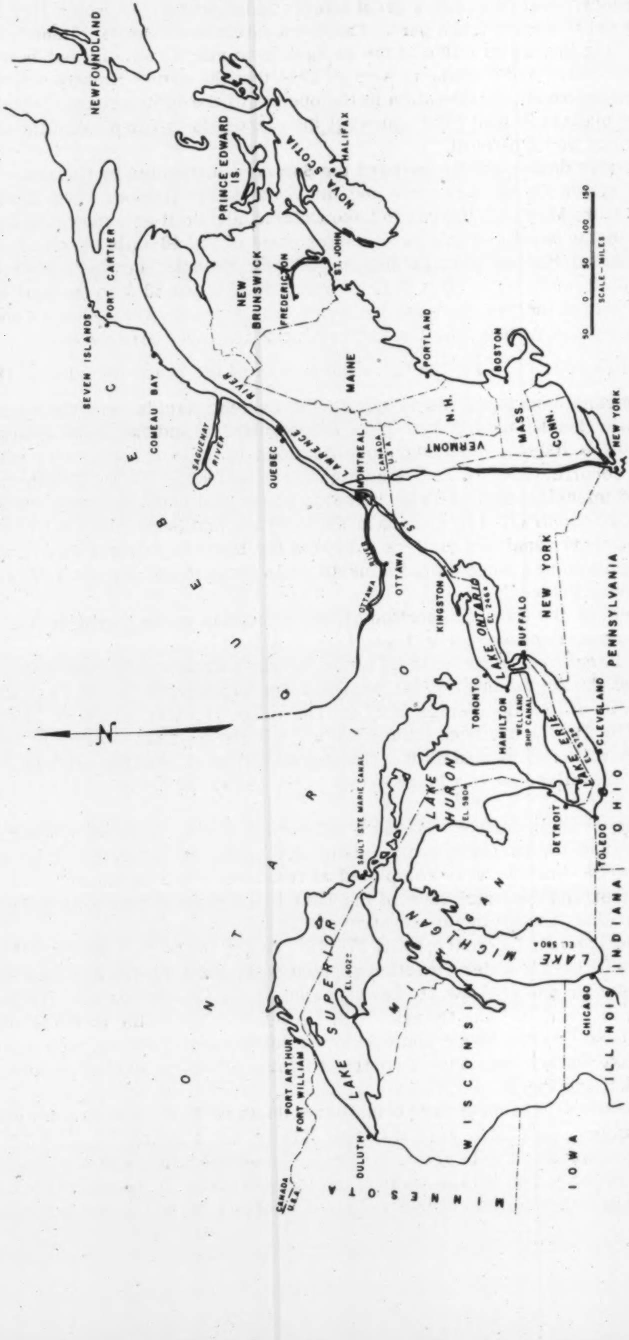
NEWFOUNDLAND
PRINCE EDWARD IS.
NOVA SCOTIA
HALIFAX
SEVEN ISLANDS
PORT CARTIER
NEW BRUNSWICK
FREDERICTON
ST. JOHN
0 50 100 150
SCALE—MILES
PORTLAND
BOSTON
MAINE
COMEAU BAY
SAGUENAY RIVER
QUEBEC
RIVER
LAWRENCE
ST.
MONTREAL
CANADA
U.S.A.
VERMONT
N.H.
MASS.
CONN.
NEW YORK
OTTAWA RIVER
OTTAWA
KINGSTON
LAKE ONTARIO
EL. 246.0
BUFFALO
NEW YORK
PENNSYLVANIA
TORONTO
HAMILTON
WELLAND SHIP CANAL
CLEVELAND
LAKE ERIE
EL. 572.0
OHIO
TOLEDO
DETROIT
LAKE HURON
EL. 580.0
SAULT STE MARIE CANAL
N
Q U E B E C
O N T A R I O
LAKE SUPERIOR
EL. 602.0
MICHIGAN
LAKE MICHIGAN
EL. 580.0
INDIANA
ILLINOIS
CHICAGO
WISCONSIN
PORT ARTHUR
FORT WILLIAM
DULUTH
CANADA
U.S.A.
MINNESOTA
IOWA

FIG. 1.—ST. LAWRENCE AND GREAT LAKES REGION

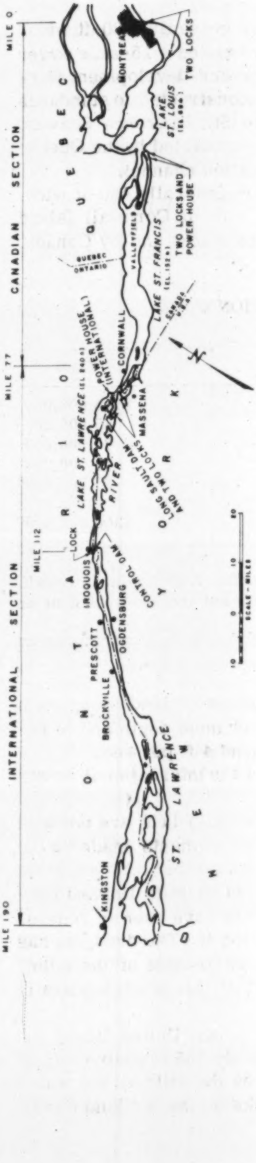
MILE 0
MILE 77
MILE 112
MILE 190
CANADIAN SECTION
INTERNATIONAL SECTION
QUEBEC
ONTARIO
NEW YORK
MONTREAL
TWO LOCKS
LAKE ST. LOUIS
TWO LOCKS AND POWER HOUSE
VALLEYFIELD
QUEBEC
ONTARIO
LAKE ST. FRANCIS
CORNWALL
POWER HOUSE (INTERNATIONAL)
LAKE ST. LAWRENCE
MASSENA
LONG SAULT DAM AND TWO LOCKS
RIVER
LOCK
IROQUOIS
CONTROL DAM
PRESCOTT
OGDENSBURG
BROCKVILLE
KINGSTON
ST. LAWRENCE
SCALE - MILES

FIG. 2.—MONTREAL TO LAKE ONTARIO

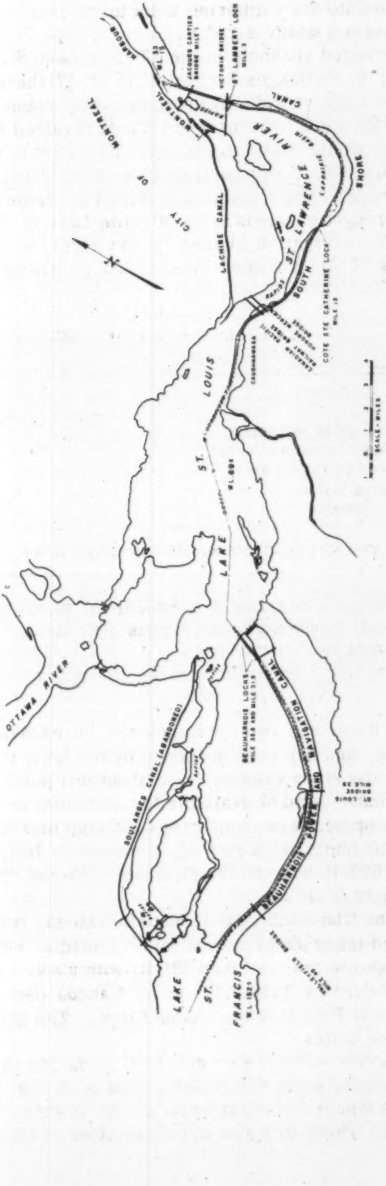
HARBOUR
W.L. 32
JACQUES CARTIER BRIDGE
VICTORIA BRIDGE
ST. LAMBERT LOCK
CITY OF MONTREAL
LACHINE CANAL
ST. LAWRENCE RIVER
LAPRAIRIE BASIN
SOUTH SHORE CANAL
COTE STE CATHERINE LOCK
CAUGHNAWAGA
LAKE ST. LOUIS
OTTAWA RIVER
SOULANGES CANAL (ABANDONED)
BEAUHARNOIS LOCKS
BEAUHARNOIS POWER AND NAVIGATION CANAL
LAKE ST. FRANCIS
SCALE - MILES

FIG. 3.—MONTREAL TO LAKE ST. FRANCIS

300 ft, which provides an extra width of channel around the long curve. The decision as to the width of this part of the channel was also influenced by the necessity to have adequate materials of suitable types for dike construction. From Côte Ste-Catherine Lock to the canal entrance (Mile 19) at Lake St. Louis the bottom width is 250 ft.

Dredged channels in the 12-mile Lake St. Louis are generally 600-ft wide.

In the Soulanges Section (Mile 31-47) there already existed a 15-mile power canal 3,000-ft wide. In the approval of plans for this power-development work in 1929, the Government of Canada required that it be constructed to standards which would make it available as a part of the future St. Lawrence Seaway. Thus, a 600-ft wide portion of the power canal has been excavated by the Quebec Hydro-Electric Commission which provides the navigation channel.

Dredged channels in the 30-mile Lake St. Francis are generally 450-ft wide.

The navigation channel in the river on the south side of Cornwall Island (Mile 77-81) has been constructed jointly by the United States and by Canada. The River has been straightened by excavation of points of land along each shore, and has been deepened by dredging to provide channel depth and to reduce the river velocity from about 8 ft per sec to around 4 ft per sec.

TABLE 1.—COST OF CANADIAN NAVIGATION WORK

Item	Cost
Lachine (with two locks)	$145,000,000
Soilanges (with two locks)	55,000,000
Iroquois (with one lock)	20,000,000
Dredging (all areas)	55,000,000
Miscellaneous	25,000,000[a]
Total (not including interest during construction)	$300,000,000[b]

[a] Includes contribution to "Power" of $9,000,000, Head Office building in Cornwall, Cornwall Bridge work, and various other items. [b] This does not include work done on deepening the Welland Canal.

Channels made available for navigation by reason of the International Power Development are in all cases not less than 600 ft.

The short channels below and above Iroquois Lock (Mile 112) are not less than 600-ft wide at the junctions with the channel improvements made by the two power entities.

The Thousand Islands Section extends from Mile 124 to Mile 190 and contained many dangerous shoals. Canadian channel widths have been in general dredged to not less than 580 ft, with about 3 miles at 450 ft. The dredging has been done by United States or Canada depending on which side of the International Boundary the channel lies. The greater part of this dredging was in United States.

Locks.—The locks built by Canada and those built by the United States are of similar basic dimensions, that is to say, 80 ft wide by 765 ft usable length from fender to breast wall, and 30-ft depth of water on the sills at low water level. These are also the dimensions of the eight locks on the Welland Canal.

Table 2 shows the mileages, and summarizes the water level ranges and lifts at the five Canadian locks.

Lock Entrance Walls.—Entrance walls to guide the ships into the locks have been layed out to suit the local site conditions, and have been designed to provide tie-up space for ships waiting to enter the locks. The objective has been to design all entrance walls to get all types of ships into the respective locks with a minimum of lost time.

Fig. 4 is a plan showing the five Canadian locks with the arrangements and dimensions of the entrance walls at each.

GENERAL LAYOUT AND CONSTRUCTION FEATURES

Lachine Section.—Over 50% of the Canadian construction work and dredging was in the immediate area of Montreal. This section has been constructed as a Seaway Project only, but with all structures located to avoid interference with a future development of the power from the 50 ft of head in this reach. The principal features of this part of the work are shown in Fig. 3. The 18.5 miles of canal from Montreal Harbor to Lake St. Louis required the excavation in the dry of 19,000,000 cu yd of common material and over 20,000,000 cu yd of rock. The two locks required a total of 870,000 cu yd of concrete. The connecting dikes required 9,000,000 cu yd of fill. In addition, a total of over 7,000,000 cu yd have been excavated by dredging at Montreal Harbor and in Lake St. Louis. The canal and its two locks were constructed by ten main contracts, two of which covered the St. Lambert and Côte Ste-Catherine Locks with adjacent parts of the channel and dike. The other eight contracts made up the remaining 15 $\frac{1}{2}$ miles of channels and dikes.

TABLE 2

Lock	Miles from Montreal	Water Level Range, Above Mean Sea Level		Approximate normal lift, in feet	Possible Limit of Lift, in Feet	
		Below lock	Above lock		Minimum	Maximum
Iroquois	112	237.5 to 244	238 to 246	$\frac{1}{2}$ to 2	$\frac{1}{4}$	18[a] 6[b]
Upper Beauharnois	32.5	108 to 112	149 to 154	41	39	44
Lower Beauharnois	31.5	68 to 75	108 to 112	41	35	45
Cote Ste-Catherine	12	35 to 38	65 to 75	35	30	40
St. Lambert	3	15 to 33	35 to 38	15 to 20	2	23

[a] During construction only, and only for 14-ft draft vessels. [b] A possible extreme condition with high level on Lake Ontario, low controlled outflow from Lake Ontario, and a low controlled operating level at the power house forelay.

Early plans for the work in this section dating from 1920 up to nearly 1950 were all based on a side canal located on the north side of the river. The continued expansion in recent years of Greater Montreal with its harbor and railway developments made it necessary to construct the Seaway on the South Shore.

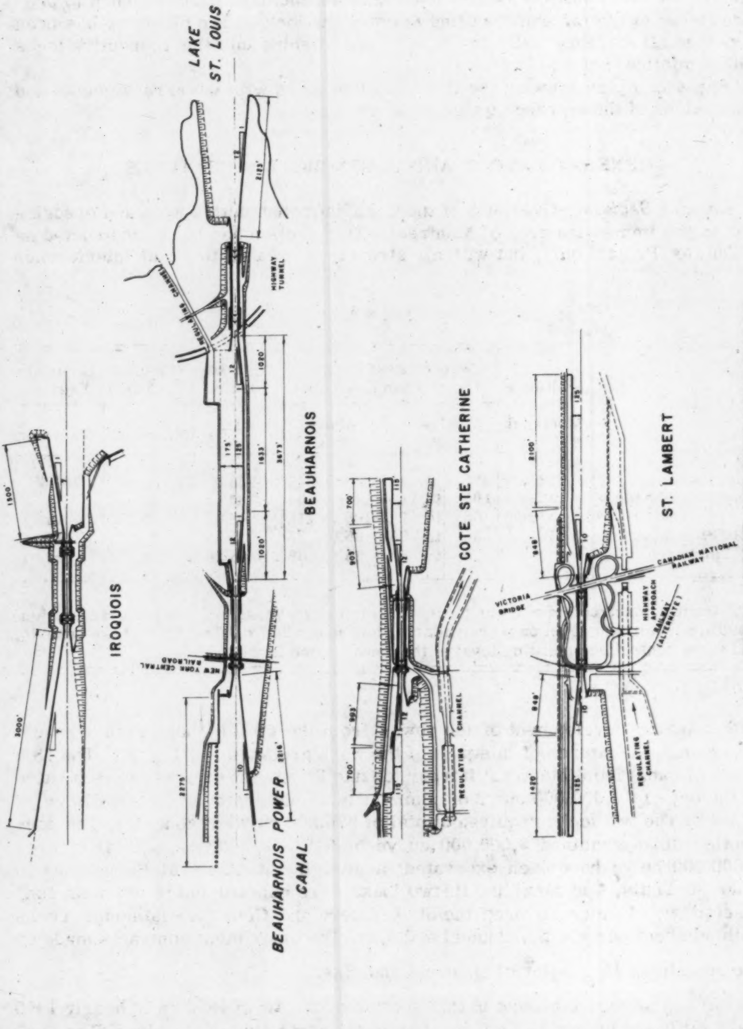
IROQUOIS
1500'
3000'
LAKE
ST. LOUIS
2123'
12
HIGHWAY TUNNEL
REGULATING CHANNEL
1020'
175'
135'
1833'
3673'
1020'
BEAUHARNOIS
NEW YORK CENTRAL RAILROAD
2275'
2216'
10
BEAUHARNOIS POWER CANAL
COTE STE CATHERINE
700'
993'
993'
700'
110
REGULATING CHANNEL
ST. LAMBERT
2100'
949'
949'
1020'
135'
10
VICTORIA BRIDGE
CANADIAN NATIONAL RAILWAY
HIGHWAY APPROACH
RAILWAY (ALTERNATE)
REGULATING CHANNEL

FIG. 4.—LOCKS AND ENTRANCE WALLS

The length of the canal became 18 $\frac{1}{2}$ miles, instead of 12 miles for the north-side site, but the problems of construction and coordination with existing facilities were much more practical and less costly. The new works in the Lachine Section are called the "South Shore Canal", as the old 14-ft canal is going to be maintained indefinitely to serve local industries. It will continue to be called the "Lachine Canal".

The entrance to the Seaway is just below Jacques Cartier Bridge (Fig. 5) and opposite the upper portion of Montreal Harbor. Three miles upstream are St. Lambert Lock and Victoria Bridge (Figs. 6 and 7). An 8-mile regulated reach extends from above St. Lambert Lock to Côte Ste-Catherine Lock (Fig. 8). The Seaway channel follows around Laprairie Basin at distances from shore varying from 1,000 ft to 2,500 ft. The actual location of the channel was selected after many comparative estimates of quantities and costs, and after many studies of suitable navigation alignment. From Côte Ste-Catherine to Caughnawaga the canal had to be located inshore, thus providing sufficient space between the Seaway and the river shore for a future enlargement of the river cross section by Quebec Hydro for the Lachine power development. Four miles above Côte Ste-Catherine Lock are the Honoré Mercier and Canadian Pacific Railway bridges (Fig. 9). Two and a half more miles of canal lead to deep water in Lake St. Louis. From just west of the Canadian Pacific Railway and around Caughnawaga village, (an Indian Reserve), the channel has again been located in the river. The cost would have been prohibitive and the number of persons displaced would have been too great to have constructed the channel overland

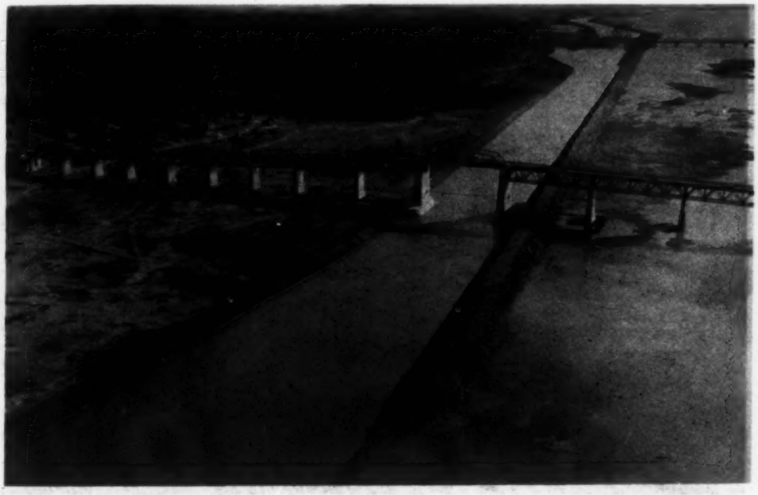

FIG. 5.—SEAWAY ENTRANCE AND JACQUES CARTIER BRIDGE

through the town. It was important to keep the restriction of the river to a minimum, to avoid affecting the water level of Lake St. Louis, and the channel therefore follows close to the shoreline.

Thirteen miles of the 18 $\frac{1}{2}$-mile canal has been constructed in the former bed of the river and the construction of the cofferdams was a major construction problem and a large item of cost. Of the ten contracts for this canal, only the three extending for 4 miles from above Côte Ste-Catherine lock to a little beyond the C.P.R. embankment were constructed on land. The six contracts below Côte Ste-Catherine were entirely constructed in the bed of the river. The cofferdams throughout this part of the work were made of materials obtained from the excavations. The contract covering over 2 miles along the river at Caughnawaga required large cofferdams constructed in fast water, for which the major part consisted of rockfilled timber cribs with steel sheet pile sheathing.

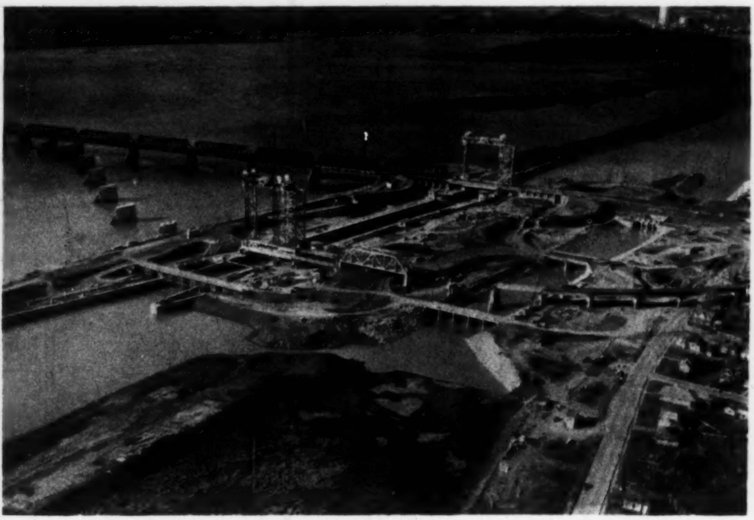

FIG. 6.—ST. LAMBERT LOCK AND VICTORIA BRIDGE DURING CONSTRUCTION

The topography of the Lachine Section required two locks to provide the lift from Montreal Harbor to Lake St. Louis with the lower lock near the City of St. Lambert, opposite Montreal, and the upper lock near the foot of the Lachine Rapids. It was impractical to split the lift into equal parts and make the two locks similar. The selected location of St. Lambert Lock immediately upstream from Victoria Bridge made possible an arrangement for uninterrupted highway and railway traffic by the use of duplicate bridges. Côte Ste-Catherine Lock was located for the most economical layout of channels and dikes.

St. Lambert and Côte Ste-Catherine Locks are both located in an area where the foundations are shale. These two locks have therefore been constructed with complete concrete floors and with the side walls designed as full gravity structures. The floor slab is nominally 3 ft thick, but is generally more due to the shale breaking along its bedding plains. Rock anchors have been used to overcome the full uplift under the slab. Fig. 10 shows the plan and cross section of the Côte Ste-Catherine Lock. St. Lambert Lock is generally similar except for the height of lift. At each of these two locks, the north wall has been designed as a gravity dam with full water level in the lock and excavated on the outside, a condition which would occur during the period of the future twinning of the locks.

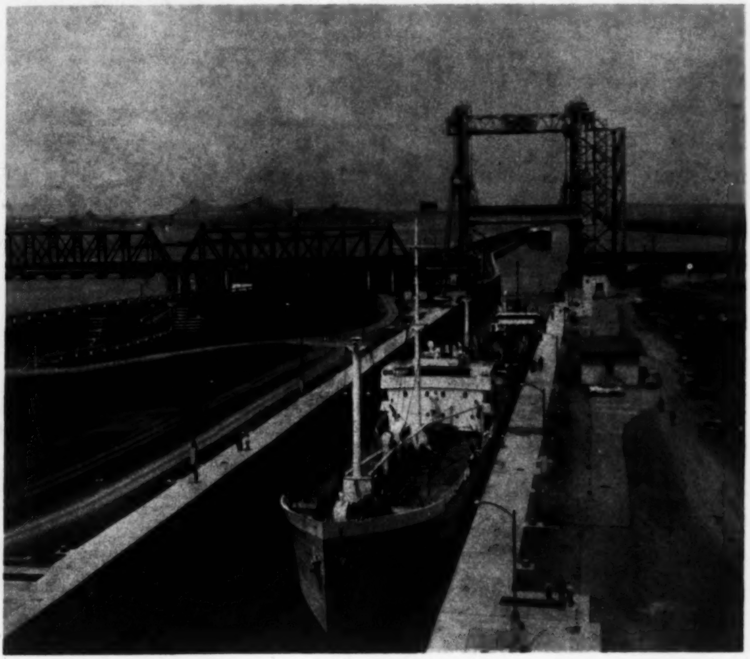

FIG. 7.—ST. LAMBERT LOCK AND VICTORIA BRIDGE

The St. Lambert and Côte Ste-Catherine locks are located in a continuous confined channel with limited space available. Consequently, the entrance walls have been constructed with the mooring portion of the wall built parallel to the canal. St. Lambert Lock has a much longer downstream wall to accommodate incoming ships.

Construction of St. Lambert Lock.—The St. Lambert Lock is located just upstream of Victoria Bridge and in the former bed of the river. The first step was the construction of a large cofferdam to unwater an area of about 150 acres.

This cofferdam had a length of approximately 1 mile parallel to the river with connections to the shore at each end. The part parallel to the river was constructed to the full section and quality required for the permanent dike of which it later became a part, which was necessary to meet winter ice conditions. The upstream shore connection was of similar construction. The downstream shore connection was of a somewhat lighter construction as no ice shoves were possible. As the schedule of lock construction required uninterrupted work throughout two winters, it was necessary to construct the cofferdam to an elevation which would make it safe against overtopping by floods caused by ice jams.

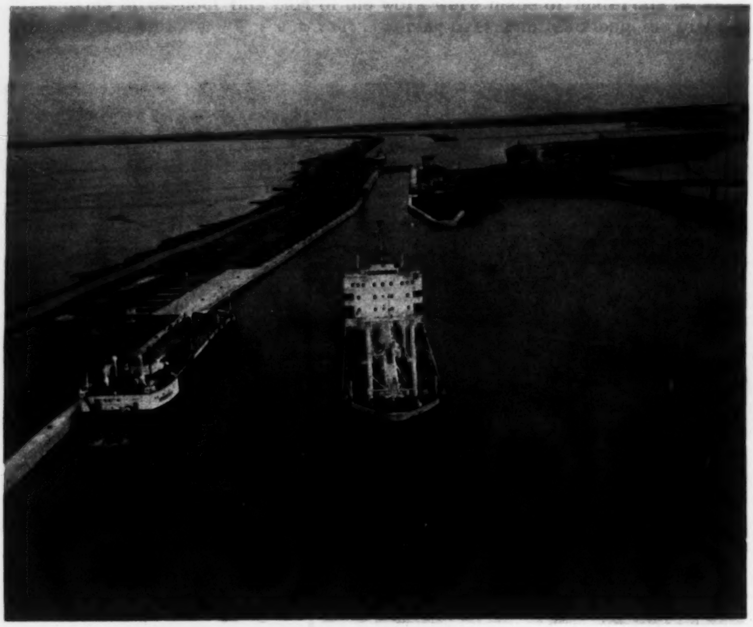

FIG. 8.—COTE STE-CATHERINE LOCK (LOOKING DOWNSTREAM)

This cofferdam was therefore constructed to El. 49, to take a 20-ft rise of water level above summer stages.

The whole cofferdam in the St. Lambert Lock area was constructed during the fall of 1955 in advance of the awarding of the general lock contract, and was turned over to the lock contractor in an unwatered condition, ready for the start of excavation. The total construction time available to the general lock contractor was 32 months (January 1956 to August 1958).

All blasting had to be conducted under very strict regulations, particularly as the excavation approached close to and under Victoria Bridge. Highway traffic on this bridge, (up to 2,000 cars per hr), had to be interrupted for every blast.

The contractor was allowed 8 months in which to advance the excavation and to start on concrete. Good production performance was made by the contractor and concrete was completed during 1957, reaching a peak rate of over 70,000 cu yd per month. St. Lambert Lock and entrance walls has a total concrete quantity of 490,000 cu yd.

The construction procedure for the strengthening and enlarging of the south abutment and the adjacent pier of Victoria Bridge was one of the most critical problems of the job. The piers of this bridge are stone masonry and were constructed over 90 yr ago, and are founded on shale, therefore it was necessary to take extreme precautions to avoid any damage. The excavation for the lock required going about 40 ft below the elevation of the river bottom shale on which the piers were founded. The first operation was the excavation of a 100-ft wide cut to grade for a distance of over 200 ft along the centerline of the lock under the bridge. The next step was the excavation of a lateral trench in shale to grade through to the abutment on the one side of the lock and to the pier on the other side of the lock. A 12-ft section of the lock structure was then constructed as a buttress against the rock underlying the pier and the south abutment. In a series of seven steps the shale was excavated up to the backline of the concrete in 12-ft trenches, and was followed after each excavation step by the construction of another 12-ft block of concrete. The whole operation of constructing the portion of the lock about 100 ft along the bridge pier and along the south abutment required a total time of almost 1 year. It was not until this

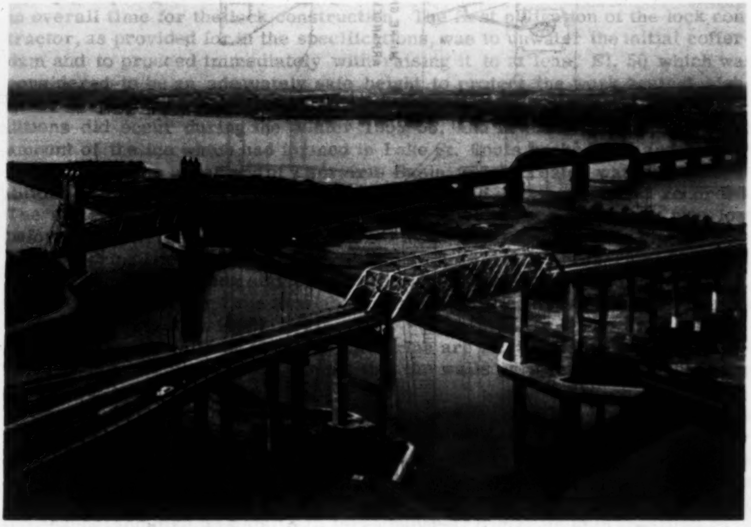

FIG. 9.—THE HONORE MERCIER AND CANADIAN PACIFIC RAILWAY BRIDGES (MILE 16)

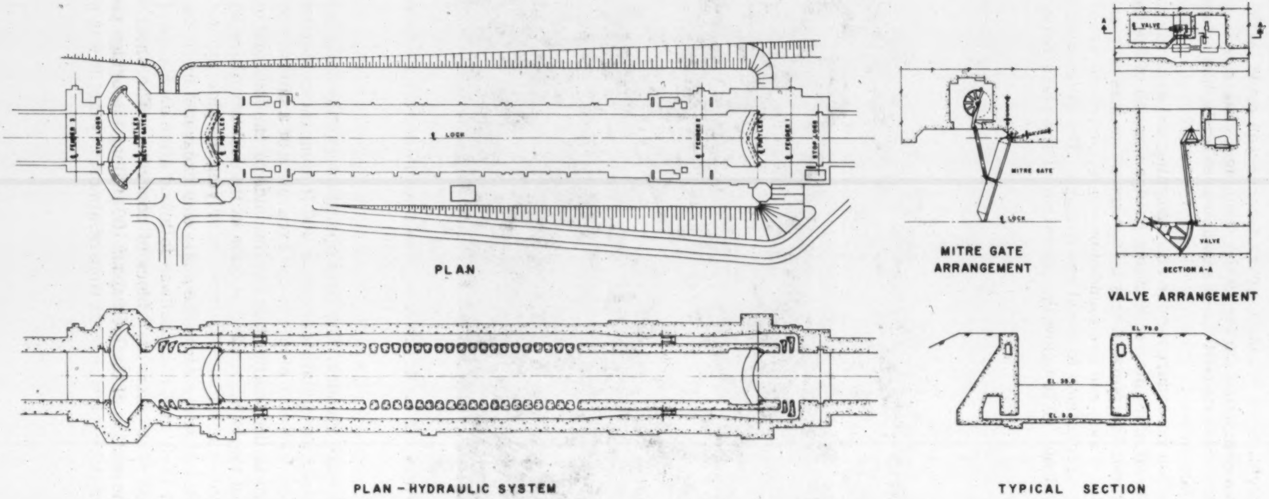
℄ FENDER 3
STOP LOGS
℄ PINTLES
SECTOR GATES
℄ PINTLES
BREAST WALL
℄ LOCK
℄ FENDER 2
℄ PINTLES
℄ FENDER 1
STOP LOGS
PLAN
PLAN - HYDRAULIC SYSTEM
MITRE GATE
℄ LOCK
MITRE GATE ARRANGEMENT
A
A
℄ VALVE
VALVE
SECTION A-A
VALVE ARRANGEMENT
EL 78.0
EL 35.0
EL 3.0
TYPICAL SECTION

FIG. 10.—COTE STE-CATHERINE LOCK

portion of the lock had been completed and made the bridge piers completely safe that any change was made in the steel work of Victoria Bridge. All during this time, railway and highway traffic passed over the bridge without interruption, except during blasting.

The mooring portion of the lower entrance wall at St. Lambert is 2,100 ft long, which will permit three large vessels to lie alongside the wall awaiting entrance, without hindering a vessel leaving the lock.

Construction of Côte Ste-Catherine Lock.—The Côte Ste-Catherine Lock is also located in the former bed of the river and the first step was the construction of a large cofferdam to unwater the area. The initial part of the cofferdam was constructed under a preliminary contract during the summer of 1955. This initial low cofferdam was turned over to the general lock contractor ready to set up pumps and start unwatering. This arrangement saved about 4 months in overall time for the lock construction. The first obligation of the lock contractor, as provided for in the specifications, was to unwater the initial cofferdam and to proceed immediately with raising it to at least El. 50 which was considered to be an adequately safe height to protect the work against high-water-level conditions in the winter due to ice jams. Extremely severe conditions did occur during the winter 1955-56, and in early February a large amount of the ice which had formed in Lake St. Louis broke away and formed an ice jam at the upper end of Laprairie Basin, which created water-level conditions at the lock site higher than any which had previously been recorded. The cofferdam was overtopped and approximately 2 months of time was lost before the cofferdam could be unwatered again and excavation resumed. Côte Ste-Catherine Lock and entrance walls has a total quantity of concrete of 380,000 cu yd. Concrete placing started in July 1956 and was completed in July 1958. The total construction time available to the general contractor was 34 months (October 1955 to July 1958).

The entrance walls at Côte Ste-Catherine are similar in general arrangement to those at St. Lambert, except that the walls parallel to the channel are only 700 ft long to take one ship.

Construction of Laprairie Basin Channel and Dike.—This channel and dike work was started in 1955, and as the work extended through parts of four seasons, the general design of the channel and dike construction and the contractor's operations had to be planned to handle the flows of three small local rivers having a combined spring run-off of about 6,000 cfs. As a result, this part of the channel and dike was split into three separate contracts with specified gaps left at two points between the contracts. The cofferdams for the Laprairie Basin work were very extensive in area, a total of about 4 sq miles of river bottom being unwatered during the final stages. The three separate sections of the channel and dike were completed independently, with the closures at the two gaps made in 1958 after the spring run-off had occurred. The main cofferdams were designed to be incorporated into the finished dike. The quantities of excavation here were roughly double the quantities required for dike construction, and made possible a reasonable amount of selection of materials. Excess excavation went into a series of islands along the inside of the channel and to reclaim low land along the shore. When the first area was unwatered during 1955, it provided adequate quantities of fill material for extending the cofferdams.

Soulanges Section.—The principal features of the Soulanges part of the work are shown on Fig. 3. The Beauharnois Power Canal (Fig. 11) and the initial power installation were constructed during the period from 1929 to 1932. The project was developed and financed by private power interests, but was subsequently acquired by Quebec Hydro Electric Commission. Approval of the work was dependent on the provision of many features which would make the canal suitable for use as part of the future St. Lawrence Seaway. Among the requirements set out by the Government at that time were the following:

1. A 600-ft channel along the north dike having 27-ft depth;
2. A mean velocity not exceeding $2\frac{1}{4}$ fps;
3. Space to be provided for future navigation locks; and
4. Bridge piers at St. Louis and Valleyfield suitable for future lift spans.

The work done in this section by the Authority to complete the Seaway included:

a. Construction of locks;
b. Highway tunnel under the lower lock;
c. Swing bridge across the upper lock for New York Central Railroad;
d. Construction of a lift span in the existing St. Louis Bridge; and
e. Construction of a lift span in the existing Valleyfield Bridge.

Consideration was given to three alternates for the 82-ft (normal) lift from Lake St. Louis to the Beauharnois Power Canal:

(1) Twin flight locks;
(2) A single high lift lock; and
(3) Two separated single locks.

The third alternative with two locks was adopted as the most desirable from the points of view of costs, reliability, uniformity of all locks and equipment, and construction time.

The lower lock was located to obtain minimum overall costs considering the variable factors of cofferdams, rock excavation, and concrete. After a series of comparative estimates, and setting the minimum desirable distance between the locks at around 3,600 ft, the position of the lower lock was fixed.

A main highway follows the south shore of Lake St. Louis, which required maintenance of traffic during construction and uninterrupted flow of traffic after the Seaway had been completed. The old highway crossed the work at about the center of the lower lock. To provide uninterrupted traffic, a four-lane twin-tube tunnel was constructed under the upstream approach to the lower lock. By completing this highway diversion as the first step in construction, traffic was kept out of the way of all subsequent work. The tunnel itself has no unusual features. It was constructed in the dry in an open cut. The stratified nature of the sandstone rock created some difficulties in sealing the tunnel.

The greater part of the lower approach excavation was made inside a cofferdam, the extent of the cofferdam having been determined by the estimated comparative costs of cofferdams and dry excavation of rock against costs of dredging rock. It was decided that the cofferdam would be constructed out into the lake to the point where rock excavation exceeding about 7 ft in depth would be done in the dry.

This lock was constructed in a total time of 32 months, of which the first 4 months provided for an initial contract to excavate rock for the highway diversion tunnel and for the construction of the Lake St. Louis cofferdam using rock from the highway excavation. The general contractor for the lock excavated 1,200,000 cu yd of rock and placed 390,000 cu yd of concrete in 28 months.

To provide the entrance to Lower Beauharnois from Lake St. Louis, it was desirable to have as broad an opening as possible for ships to enter the protected approach area. Consequently, the wall for this entrance extends in a straight line for a little over 2,100 ft on a flare of 1:12. At the outer end of the entrance wall, the water in Lake St. Louis is over 50 ft in depth. This part of the wall consists of twelve reinforced concrete cribs, each 115 ft long, founded on a prepared base of heavy rockfill. The tops of the cribs were designed to be just above normal water level, which provided for an 8-ft cast-in-place concrete coping on top of the cribs.

The pool between the two Beauharnois locks has a continuous wall along the south side.

The New York Central Railroad crosses the Beauharnois Power Canal on a 3,000-ft long bridge. It was desirable to locate the upper lock either just above or just below the existing New York Central Railroad crossing, in order to use a movable railroad span with an 80-ft gap, as compared to a much longer span if the bridge were separated from the lock. It was decided to locate the lock below the bridge for the following reasons:

FIG. 11.—BEAUHARNOIS POWER CANAL QUEBEC HYDRO POWER DEVELOPMENT (2,2000,000 HP) AND THE TWO BEAUHARNOIS LOCKS

(a) A longer entrance channel from the power canal, and therefore a greater distance in which ships can reduce speed;

(b) Reduced effect of any interference between trains and ships. Ships will always be able to pass between the upper and lower locks; and

(c) Lower cost.

One of the site problems was the maintenance of railway traffic during the construction of the lock and bridge. A work program using two stages of diversion was adopted. The first diversion permitted the construction of a short length of lock walls at the location of the existing railway. The second diversion crossed the completed lock walls section, just clear of the future swing bridge. This permitted the remainder of the lock excavation and concrete to proceed with no further hindrance from the railroad.

This lock was constructed in a total time of 29 months. The major quantities included 1,000,000 cu yd of common excavation, 1,400,000 cu yd of rock excavation and 330,000 cu yd of concrete.

The entrance to Upper Beauharnois Lock from the power canal required that no serious obstruction be imposed on the flow to the existing power house. The south wall for this entrance extends for a little over 2,200 ft in a straight line on a flare of 1:10. Because of the current in the power canal, and the long distance from a good anchorage, a 2,200-ft snubbing wharf has been constructed along the north side of the entrance to the lock. This is long enough to accommodate two large vessels awaiting entrance to the lock.

A major problem of construction for the two Beauharnois locks related to the large amount of work going on at one time by The Seaway Authority and by Quebec Hydro. A total of over 4,000,000 cu yd of hard sandstone rock was excavated by four contractors (three for the Authority and one for Quebec-Hydro) in the one congested area. The problem was made more difficult by the many existing high tension transmission lines which had to be protected and relocated.

Canal and Lock at Iroquois (Fig. 12).—This lock by-passes the Control Dam constructed by the New York State Power Authority. One of the many features of cooperation between power and navigation was the necessity to complete the Iroquois Lock in early 1958 to permit the abandonment of parts of the existing 14-ft canal in order to permit the flooding of the new power pool.

A 3,000-ft entrance wall has been constructed above the lock in order to provide a safe mooring place for at least three large downbound vessels. A 1,500-ft wall has been constructed for the downstream entrance.

There were no particular construction problems at Iroquois other than the excavation of 4,500,000 cu yd of dense glacial till and the placing of 320,000 cu yd of concrete.

Hydraulic Design of Locks.—Iroquois Lock is filled and emptied by the operation of the main lock gates (sector type).

The four downstream Canadian locks have all been constructed with side culverts and lateral ports, with the flow for filling and emptying controlled by sector-type valves at the upstream and downstream ends of the culverts. The valves (12 ft by 14 ft) are identical for all locks, and the culverts and ports have essentially similar cross-sectional areas. The actual culvert shapes and the size and number of lateral ports have been varied to suit the structural shape of the different lock walls. The hydraulic details were developed by use of a 1/30 scale model.

At St. Lambert and Côte Ste-Catherine, the lock walls are complete gravity structures built on excavated shale foundations. The culverts are 16 ft 6 in. wide at the base by 20 ft. 0 in. high. The culvert is shaped as shown on Fig. 10. There are twenty lateral ports in each wall located in the middle 400-ft length of the lock. The ports are all 2 ft 7 in. wide by 3 ft 7 in. high.

At the two Beauharnois locks, the walls have been constructed in sound durable sandstone and the wall thickness has been governed mainly by the space requirements for the culverts. To maintain essentially similar hydraulic characteristics as for Côte Ste-Catherine and St. Lambert, the culverts at Beauharnois have been made narrower and higher. The concrete wall separating the culvert from the lock chamber has also been made narrower here.

The culverts are 13 ft 6 in. wide at the base by 25 ft 0 in. high. There are forty lateral ports in each wall located in the middle 400-ft length of the lock. The ports are 1 ft 3 $\frac{1}{2}$ in. wide by 3 ft 6 in. high.

Table 3 shows some of the dimensional characteristics of the lock hydraulic systems. All figures relate to one wall. The other wall is similar.

Under the maximum head and discharge valves full open conditions at Beauharnois, the flow in each culvert will be around 6,000 cfs.

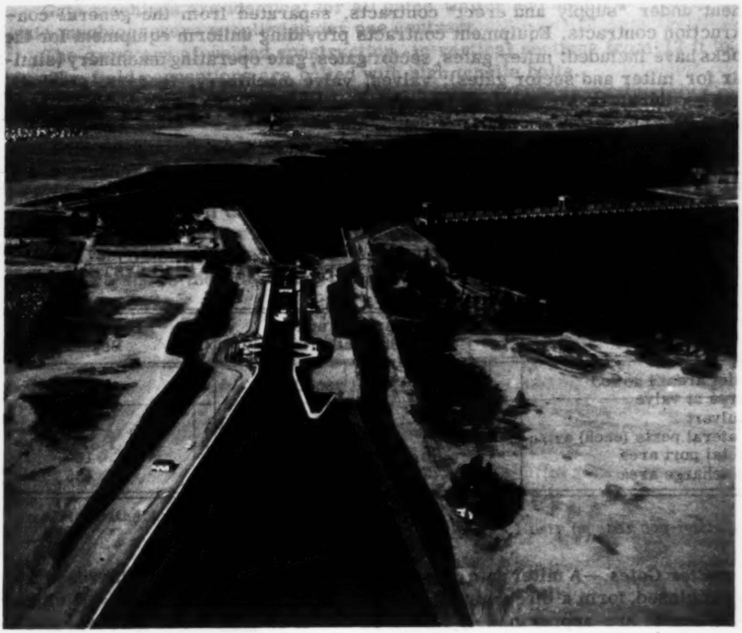

FIG. 12.—IROQUOIS LOCK AND IROQUOIS CONTROL DAM

The calculated operating times for the Beauharnois locks, based on the hydraulic model tests were:

Fill	6.5 min
Empty	7.3 min

Actual operating results have shown that for a 41-ft lift, and with a valve movement time of 2 min (fill) and of 1 min (empty) the possible times are approximately:

Fill	6.5 min
Empty	6 min

The operating times for the other locks are less, and are roughly proportional to the head.

Up to the present time (1960), for precautionary reasons, the locks have been operated to fill and empty in times varying from 6 min to 12 min while operators are becoming familiar with the hydraulic characteristics.

LOCK EQUIPMENT

Canadian policy for the Seaway construction has been to provide all equipment under "supply and erect" contracts, separated from the general construction contracts. Equipment contracts providing uniform equipment for the locks have included: miter gates, sector gates, gate operating machinery (similar for miter and sector gates), valves, valve machinery, fenders, stoplogs, stiffleg derricks for stoplogs, de-watering pumps, line haulers, and diesel generator units.

Electrical equipment has been generally standardized throughout all locks. The motors for all of the gates and for all of the valves and the fenders are of the same size and characteristics. All of these motors were purchased under one contract.

TABLE 3.—AREAS, IN SQUARE FEET, OF LOCK HYDRAULIC SYSTEMS

Item	St. Lambert and Cote Ste-Catherine	Lower and Upper Beauharnois
Inlet area	504	504
Area at valve	168	168
Culvert	300	305
Lateral ports (each)	9.25	4.5
Total port area	185	180
Discharge area	427	435

Miter Gates.—A miter gate consists of two horizontally framed leaves which, when closed, form a three-hinged arch against the hydrostatic load. All miter-gate leaves are approximately 5 ft thick and 46 ft wide. The gate leaf in the closed position is at an angle of 1 in 3 to the face of the lock.

The horizontal girders in a leaf terminate at each of the vertical leaf edges in the quoin and miter posts. The quoin post forms the edge next to the lock wall. The miter post forms the edge next to the lock center line when the leaf is in a closed position. The bottom hinges on which the gate leaves turn, are pintles, identical in every case, of 21-in. diameter. The upper hinges are of a simple pin type and are also identical in every case. Each leaf has a skin plate on the upstream side only, the other side being left open for economy and ease of maintenance, but has adjustable diagonal members. The adjustable diagonal members when prestressed give the gate leaf its required additional stiffness for operation. All lock gates are poly-vinyl coated for protection against corrosion.

The pintle assembly consists of a heel casting, which is attached by means of tuned bolts to the bottom of a gate leaf at the quoin end, a phosphor bronze bushing inserted in the heel casting, a nickel steel hemispherical pintle on which the bronze bushing bears and a base casting embedded in concrete which supports the pintle and gate leaf.

The miter gates have been designed to withstand the hydrostatic loads and to be operated against a differential head of 0.50 ft. The dead-load forces are those which result from the weights of the gates.

The gates are operated by a horizontal sector gear with a strut attached near the top of the gate. The machine is designed to exert a thrust of 1,480 ft-kips through a spring-loaded strut. The gate machine arrangement is shown in Fig. 10. The motors are wound-rotor type of 20 hp.

Gate machines are identical for all gates (miter and sector). Thirty-four gate machines were supplied under one contract.

The gates are of welded construction, in vertical sections from 12 ft to 14 ft. The field connections are bolted with high tensile bolts.

The normal time for opening or closing a miter gate is approximately 2 min. Miter gates installed at the downstream end of a lock may be subjected, at times, to a severe back surge either from dumping the lock, or from ship propellers if the gates are being closed immediately after a downbound vessel leaves the lock. Provision has been made to release such a gate from the operating machine if the force due to the back surge exceeds a strut load of 140,000 lb plus or minus 10%, when the gate is near the mitering position. Release is accomplished by the shearing of pins in the gate machinery.

Table 4 shows the miter gate installation at each of the locks.

TABLE 4

Lock	Upper End	Lower End
St. Lambert	1 pair 1 pair (space non-operating)	2 pairs
Cote Ste-Catherine	1 pair	1 pair 1 pair (spares non-operating)
Lower Beauharnois	2 pairs	2 pairs
Upper Beauharnois	1 pair	1 pair
Iroquois	none	none

The weights of the individual gate leaves are:

All upper gates - 132 tons
St. Lambert Lower Gates - 175 tons
Côte Ste-Catherine Lower Gates - 238 tons
Beauharnois Lower Gates - 258 tons

A floating gate lifter having a lift capacity of 300 tons is being designed. If a gate should be damaged before the delivery of the lifter, and this is a calculated risk, temporary rigging will be required.

Sector Gates.—Each sector gate consists of two vertical sectorial prisms. The radius of each sector is 48.7 ft, with a 43° angle included between its radial members and having a height of about 43 ft. The sector framing consists of seven horizontal structural steel sector frames. The skin plate on the outer faces of the curved steel girders forms the curved vertical face of the prism. The outer faces of the radial horizontal frames form the two vertical faces of the prism. The two outer vertical edges of the curved face extend approximately 8 ft beyond each radial vertical face. The two radial faces of each sector are of open framing with vertical and horizontal members, except for the ice guard gratings, the diagonal braces and the plate girders at the bottom of these faces. A vertical girder of V-section extends the full height of gate from heel casting to top anchorage. This girder transfers all the water load to pintle and top bearing. The heel casting is fitted with a bronze bushing which turns on a nickel steel 30-in. diameter pintle. The pintle is set in a base casting which in turn is connected to an anchorage which extends back into the concrete monolith.

The weight of each sector is supported at three main points—on a pintle beneath the junction of the two radial faces and on the two roller trucks which are beneath the outer ends of the radial faces. The rollers travel on a curved steel track and are close to the skin plate. In addition to the three main points of support there is a hinge at the top of each sector, directly above the pintle. The top hinge and the pintle have embedded anchorages which are designed to withstand the entire horizontal loads.

The sector gates are operated by operating machines identical with those provided for the miter gates. The weight of each sector is 246 tons.

Iroquois Lock has four pairs of sector gates, two pairs at each end, Upper Beauharnois has one pair, and Côte Ste-Catherine has one pair. At Iroquois the four pairs of gates are completely equipped with operating machinery so that, in case of one gate being damaged, the adjacent pair can be immediately brought into use. They serve as lock-operating gates and also as the means of filling and emptying the lock. For a three-month period during construction of the power project, it was necessary to operate these Iroquois sector gates under a head of 18 ft. The operation under those head conditions was smooth and uneventful with a filling flow of around 3,000 cfs. Permanent operating conditions will be under a head range of from $\frac{1}{2}$ ft to 2 ft and occasionally a head of up to 6 ft.

The sector gates at Upper Beauharnois and Côte Ste-Catherine have been provided as emergency gates, which can be closed against a flow of water in case of any accident to the downstream miter gates. All sector gates have been designed for a hydrostatic head of 40 ft, which can occur at Upper Beauharnois or at Côte Ste-Catherine with the upper miter gates open and the lock at its low water level. The regulations established for operating procedure require that the operator starts closing the sector gates promptly after the stern of a downbound ship passes these gates. If a ship should crash through the fender and knock out the downstream gate, the damage would be limited to the gate, the fender, and the vessel. The water suddenly released would be little more than the volume contained in the lock, as the sector gate would quickly close off the flow at the upper end of the lock.

Lock Valves.—The sixteen filling and emptying valves for the four locks (excluding Iroquois) are identical and interchangeable except for the operating strut arms. All valves were supplied and erected under one contract. The weight of one valve is 25 tons. The valves are 12 ft wide by 14 ft high and can be opened or closed in 1 min. The bottom seal is steel on babbit. The side and lintel seals are rubber. Each valve is set in a well extending down from the coping, large enough so that a complete valve can be removed and replaced in case of a breakdown. Two spare valves are on hand.

Fig. 10 shows the general arrangement of a valve and its operating machine. The strut arm has a spring mounting having a total compression of 8 in. The strut arm and machine can exert a pull of 166,000 lb. The motors are 20 hp.

Wire Rope Fenders.—Wire rope fenders are installed on each side of the lower lock gates for the purpose of protection against ships which have travelled beyond the prescribed limit. Fenders are also installed upstream from the upper gates at all locks, and downstream from the upper gates at St. Lambert and Iroquois. At the other three locks, the breastwall protects the downstream side of the upper gates. If a ship hits the fender its bow will come in contact with a three-part double system of $1\frac{3}{4}$-in. diameter wire ropes stretched across the lock. These ropes are reeved through sheaves anchored to bollards encased in the lock wall. As the ship continues to advance the ropes unwind from two drums against four friction brakes gear-connected to the drums, thus maintaining a steady rope pull. The kinetic energy from the moving ship is dissipated through the ropes sliding across and along the bow, over a friction bollard just ahead of the drum, and through the drum brakes. The fenders have been designed to absorb the energy of a ship weighing 40,000 tons moving at a speed of 3 mph through a distance of 68 ft.

Two types of machine elements comprise the complete fender machinery. The exposed machine elements, the sheaves, the boom, the bascule, the boom latch, the bascule frame with driving screws, the bollards, and that portion of the rope which stretches across the lock, are outside on top of the lock walls. The housed machinery elements, the rope drums with brakes, the rewinding and brake scouring machine and the bascule drive unit are inside in a machine room within the lock wall.

Stoplogs.—Any of the five Canadian locks can be completely unwatered in case of emergency or for maintenance by placing the stoplogs which are provided at each end of all locks. There are two types of steel stoplogs (80-ft clear span) standard for all locks, and a third type required for the lower end of St. Lambert only. They are all of approximately the same weight, a little over 30 tons, but vary in height and strength to suit the range of head conditions. All stoplogs have two rollers at each end to permit placing in flowing water. The stoplog gains are located at the ends of the lock to permit unwatering of the entire lock area in which all of the operating equipment is located. The stoplogs are not heated and require a portable steam generator for winter handling.

Permanently installed stiff-leg derricks at each end of all locks place and remove the logs. The derricks are rated at 55 tons at 55-ft radius and have 75-hp motors.

The heights, weights and distribution of the three types of logs are shown in Table 5.

Unwatering Pumps.—Each lock can be dewatered within an interval of from 15 hr to 18 hr by means of the permanent pumping installations. St. Lambert, Cote Ste-Catherine, Lower Beauharnois and Iroquois Locks are each equipped with two vertical 24-in. propeller type pumps capable of discharging an average total of around 40,000 gpm. Upper Beauharnois Lock has no dewatering pumps, but can be drained by gravity through the lower lock. All eight pumps are similar except for the length of the riser and shaft, and they were purchased and erected under one contract. They are driven by 250-hp motors.

Line Haulers.—The four locks (excluding Iroquois) have line haulers spaced along the side of the lock wall on which the ship is moored. These line haulers are provided to facilitate the handling of wire ropes from the ship up to the coping and are particularly important for upbound ships where the height from the deck of the ship up to the coping is of the order of 40 ft.

Power Supply.—The main power supply at all locks is furnished by the public utility in the area. At. St. Lambert, Côte Ste-Catherine and Iroquois locks, and at all of the lift bridges, there are individual diesel electric units for use in cases of emergency. The size of the diesel electric units at the locks is 300 kva.

TABLE 5.—STOPLOGS

(a) Types

Type	Height of log	Design head, in feet	Remarks
A	3 ft 8-7/8 in.	43	Bottom group, all except lower end of St. Lambert
B	4 ft 10-1/4 in.	24	Top group, all locks
C	3 ft 8-7/8 in.	59	For lower end of St. Lambert only, using high-tensile steel

(b) Distribution

Lock	Type A	Type B	Type C
Iroquois	10	10	. . .
Upper Beauharnois	10	9	. . .
Lower Beauharnois	10	10	. . .
Cote Ste-Catherine	10	9	. . .
St. Lambert	5	7	12
Total	45	45	12

BRIDGES

Brief notes relative to the modification of existing bridges or construction of new bridges follow.

Jacques Cartier Bridge (existing, Mile 1).—This is the most important highway bridge (five traffic lanes) from Montreal to the South Shore. It was built in the 1920's, and has a high-level crossing over the north channel. The high clearance permitted access to the upper part of Montreal Harbor and also to the proposed entrance to the Seaway which at that time was planned to be located along the north shore. The part of the bridge south of St. Helen's Island sloped at an easy grade down to the south shore. At the Seaway channel location the bottom of the steelwork of the bridge was only 40 ft clear of the high-water level in the river. In order to avoid a lift span, 120 ft clearance has been obtained by jacking up the bridge by 50 ft, and by replacing one deck span by a new through truss to gain an additional 33 ft. The new truss was erected on falsework on the upstream side of the bridge, and other falsework was constructed on the opposite side for dismantling the old truss. The only complete traffic interruption was one of about 6 hr in October 1957 when the old span was shifted out and the new one moved in.

Victoria Bridge (existing, Mile 3).—This bridge carries the double track main line of the Canadian National Railways and is an important highway crossing. As it was impractical and would have been of very great cost to carry the railway over the Seaway at a high level, and a tunnel would have been of even greater cost, a lift span had to be provided. There were strong objections from the public about the highway-traffic interruptions which would occur, and the Canadian Government concluded that arrangements must be made for uninterrupted highway traffic over this bridge. This decision was a major factor which influenced the precise location of St. Lambert Lock. By locating the lock immediately upstream from the bridge, it was possible to provide for uninterrupted highway traffic by having duplicate bridges, one at each end of the lock. Normally, all traffic uses the downstream lift span (which is the south span of Victoria Bridge). When this span is raised for an upbound ship entering the lock or a downbound ship leaving it, all vehicular traffic will be diverted to the other bridge. After this plan had been adopted for the highway traffic, the Canadian National Railway decided that a similar arrangement of alternate routes should be provided to permit uninterrupted railway traffic. This railway diversion bridge will be completed in 1961.

Victoria Bridge - Alternate (new, Mile 3.2).—A vertical-lift span has been constructed across the upper end of St. Lambert Lock to provide the alternate route during periods when Victoria Bridge is raised.

Côte Ste-Catherine (new, Mile 12).—A rolling lift bridge has been constructed across the upper end of Côte Ste-Catherine Lock for access to the dike. It has cast steel segmental and track girders. It is a service bridge only.

Honoré Mercier (new approaches to existing bridge, Mile 16.4).—This is an important highway crossing. The old approach ramp has been demolished, and an entirely new high level approach structure has been built.

Canadian Pacific Railway Bridge (new, Mile 16.6).—This bridge carries the double-track main line of the Canadian Pacific Railway. Two single-track vertical-lift bridges have been constructed close to the former embankment. The bridges were constructed complete on a new alinement about 100 ft east of the existing embankment, and after diversion of the railway onto the bridges, the old embankment was excavated. The use of two single-track bridges will permit mechanical maintenance on one bridge in the raised position, with negligible interference to railway or Seaway traffic.

New York Central Railroad (new, Mile 33).—A bob-tail swing span has been constructed across the upper end of the Upper Beauharnois Lock. It is also used as a service bridge.

St. Louis Bridge (existing, Mile 39).—This is a combined railway (Canadian National) and highway structure across the Beauharnois Power Canal. Two old 100-ft fixed-girder spans have been replaced by a new 200-ft vertical-lift span weighing 1300 tons. One old pier has been removed.

Valleyfield Bridge (existing, Mile 45).—This is a combined railroad (New York Central) and highway structure across the Beauharnois Power Canal. Two old 100-ft fixed-girder spans have been replaced by a new 200-ft vertical lift span weighing 1,480 tons. One old pier has been removed.

It is of interest to note with reference to the foregoing two bridges that when the Beauharnois Power Development was constructed (initial stage completed during 1929-1932), the Canadian Government required that piers be constructed in the bridges across the canal at Valleyfield and at St. Louis suitable for future lift spans. For use up to the time of completing the Seaway, a temporary pier in the middle of the lift span gap and two 100-ft spans were constructed. The recent modification has required the building of a new 200-ft span with towers and lifting machinery at each bridge, and the removal of the old spans and temporary piers. The new Valleyfield span was assembled on shore $\frac{1}{2}$ mile from the bridge. On a selected day in September 1958, the old short spans were removed by floating out on scows, and immediately following, the new span was floated into place on two other large steel scows. In early April 1959, the temporary pier weighing 750 tons was broken off at the ship channel grade, and the top part lifted up by two scows using four of the salvaged plate girders as a strong-back between the scows, and floated away from the ship channel. The St. Louis Bridge was handled in a similar way. At both of these bridges, the Seaway Authority has built the new lift spans, and Quebec Hydro has removed the old spans and temporary piers.

Cornwall Island Bridges - South and North Channels (new, Mile 81).—The piers for the South Channel Bridge were constructed by Canada. The superstructure was constructed by United States. The construction of the bridge across the North Channel is just starting (1960).

Iroquois (new, Mile 112).—A rolling lift bridge similar to the one at Côte Ste-Catherine has been constructed across the lower end of the Iroquois Lock. It is essentially a service bridge for use by the Authority and for access to the north end of the Iroquois Control dam.

LOCK OPERATIONS

The major factor limiting the capacity of the Seaway is the practical number of lockages which can be made in a day. The lockages per day are governed by the physical characteristics of the locks (hydraulic, mechanical and electrical), and by the speeds at which ships can be got into and out of the locks.

The lock physical characteristics consist of two basic elements:

1. The times required to open and close the gates and fenders; and
2. The times required to fill or empty the lock.

At the Canadian locks, the gates and fenders are electrically interlocked so that they operate in sequence.

The problem of getting ships into and out of the locks safely and quickly relates to the ships and their captains, to the lock-entrance arrangements and facilities, and also to the weather conditions.

The present operating time cycle under good weather conditions for the Canadian locks ranges about as follows:

Ship entering lock	15 to 25 min
Close fenders, then gates	3 min
Fill or empty lock	7 to 8 min
Open gates then fenders	3 min
Ship leaving lock	8 to 12 min
Total	36 to 51 min

The present average time cycle under good weather conditions is about 45 min, or about 32 lockages per day could be made if ships arrived steadily and uniformly at all locks.

Some improvements are possible in each of the foregoing time factors, but the greatest variable and the most rewarding place for gain is the time to get a ship into the lock. This feature involves good dispatching and clear instructions for the ships' captains, as well as providing adequate approach facilities.

Operations to this date (1960) have shown up a number of the critical spots which could influence traffic capacity. For example:

a. Ship movements at St. Lambert Lock are restricted by the many trains crossing Victoria Bridge (over 100 trains per day). These restrictions will be eliminated when the alternate railway bridge has been completed in 1961.

b. The Beauharnois locks require careful dispatching and good coordination between lockmasters and ships' captains. Because of the short distance between these locks, one slow ship will affect both locks, and the passage of one slow ship could cause twice the delay here compared to a delay at a single lock.

Traffic.—Traffic through the Seaway to the end of August, 1959 amounted to a little over 11,000,000 tons in about 2,000 ships upbound and 1,900 ships downbound. Of the total of about 3,900 ships passing through the locks, approximately 35% have been "Ocean" and 65% have been "Inland" type.

Maximum capacity of the Seaway is influenced greatly by three major factors entirely apart from the characteristics of the locks themselves. These three are:

(1) Uniformity of cargo traffic throughout the 8-month season;
(2) Reasonably equal upbound and downbound cargoes; and
(3) Reasonable proportion of large Lake vessels.

The Welland traffic up to 1958 never exceeded 24,000,000 tons of cargo per yr. This can be greatly increased. A total of 60,000,000 tons of cargo per yr through the St. Lawrence Seaway is possible if the foregoing three factors are reasonably attained.

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TRANSACTIONS

Paper No. 3164

AIRPHOTO INTERPRETATION FOR AIRFIELD SITE LOCATION

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SYNOPSIS

The principles and procedures of airphoto interpretation are briefly considered. To illustrate the application of airphoto interpretation to airfield site selection, an area (Martinsburg, W. Va.) has been selected and an airfield-site-location analysis prepared. From a study of the airphotos, three sites were selected for preliminary study and a comparative analysis is presented.

INTRODUCTION

The use of aerial photography in engineering planning is not new; many engineers and engineering firms use photogrammetry to obtain the required topographic mapping of a given area at the most economical cost. Information is now fed from the plotter into an electronic computing machine to come up with earthwork quantities. These engineers will be quick to claim that they have adopted modern methods and can do a better job at less cost, but have they made complete utilization of this tool, aerial photography? Many engineers have not.

There are engineers that are becoming aware of another use for aerial photography. This group is aware of the fact that before designing a major engineering project there is need of a well-planned comprehensive study of all factors that will effect the cost of the project. They realize that early in the

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planning phase one must gather regional information and then particular information about all possible sites. Then a comparative study of these possible sites can be made.

Aerial photography is an efficient tool for study of the regional geography and physiography. It is the most efficient way of studying two or three sites in detail for use in preparing a comparative study. That is, it can be used during the planning stage to make an analysis of the soils and other environmental factors that will provide the basis of comparison between alternate sites. Many factors that affect the planning, design, and construction of an engineering project may be determined by a careful study of the airphotos.

The use of airphoto-interpretation techniques to evaluate soil and materials is not new, but has been developed over a period of 15 yr to 20 yr. This has been largely on a research basis, but a few progressive highway organizations and contractors have used such methods for several years with excellent results.

The purpose of this paper is to discuss airphoto interpretation and its application to airfield engineering. First the principles and procedures will be discussed and then an area will be analyzed as an example of the procedure and the information that can be gained.

APPROACH

Principles.—The principle of airphoto interpretation has been explained many times and will be cited only briefly herein. Several references in the bibliography explain the principles and the techniques in detail. The basic principle of the technique is that like materials under similar topographic and climatic environments will exhibit similar airphoto patterns, and unlike materials will present different patterns.

The pattern can be described as the combination of certain recognizable elements as recorded upon the airphoto. These elements that singly or together form a significant pattern are: landform, drainage system, erosion characteristics, photo tones, vegetation, and land use. The basic principle and pattern concept are illustrated in Figs. 1 and 2. Similar materials under similar climatic conditions, but separated by a distance of about 100 miles, are shown in Fig. 1; the two photographs represent patterns of residual soils over limestone (note the similarity in landform, drainage, and land use). Dissimilar materials under similar climatic conditions in southern Indiana provide contrasting airphoto patterns as seen in Fig. 2. Fig. 2(a) illustrates the airphoto pattern (sinkhole pattern) developed in areas where flat-lying limestone and its residual soils are exposed to a semi-humid climate. Here the soils, consisting of plastic clays, will be found to be 6 ft to 10 ft deep and only infrequent, shallow rock excavation will be necessary for airport construction. Fig. 2(b) illustrates another rock type in this area, that of massive sandstone beds. The topography consists of flat, blocky upland areas with steep valley walls. The soils are shallow upon the uplands. Fig. 2(c) illustrates the airphoto pattern developed in areas of flat-lying shales in the same general region. Rolling topography, intricate drainage, and erosion pattern indicate residual soils over shales. Here the topography is too rolling to locate an airport.

Procedure.—To provide a well-organized analysis of an area for airfield planning or any similar project, the engineer must proceed in a systematic manner. This means starting with the known and working towards the unknown:

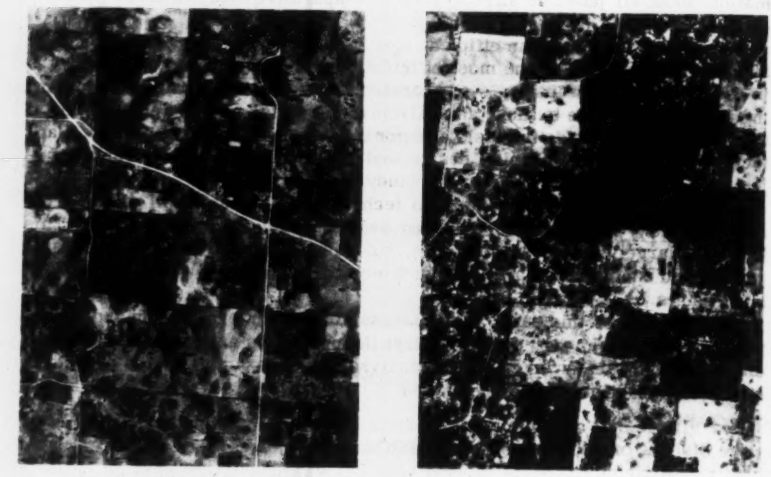

FIG. 1.—SIMILAR MATERIALS CREATE SIMILAR PATTERNS

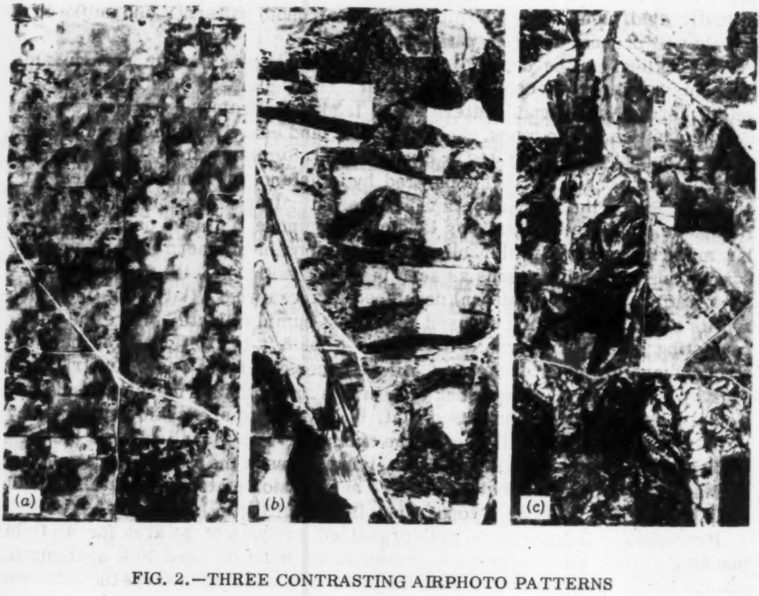

FIG. 2.—THREE CONTRASTING AIRPHOTO PATTERNS

that is, working from the general to the particular. Photo interpretation requires the ability to select the significant details from a vast array of details. The engineer doing the analysis or interpretation should have an adequate background in soils, geology, and other earth sciences to enable him to understand a sequence of events or to evaluate the significance of features that individually or as a part of a system provide the information required to make a correct analysis.

Briefly, the logical procedure to follow is: (1) assemble a mosaic of an area somewhat larger than the area of concern, (2) study this mosaic and delineate the major natural and cultural boundaries of the landscape, (3) study stereoscopically the individual prints to determine the details within the larger areas already delineated, and (4) prepare soil and drainage maps and prepare a report on the conditions which affect the cost and the design of the project.

Engineering Use of Information.—Airphoto interpretation will save time and provide better planning in several ways. Several uses of the techniques are:

1. Preliminary interpretation of soils and other natural environmental factors that influence the planning and location of a project.
2. Engineering soils maps of proposed locations.
3. Location of construction materials.
4. Determination of potential engineering problems, such as hard rock cuts and unstable soil conditions.

It should be emphasized that, to obtain the most from airphoto interpretation, it should be utilized at the earliest stages of planning. Airphotos provide the engineer the opportunity for an areal concept of large areas during the planning stages.

AREA ANALYSIS

To illustrate the application of the technique to airfield engineering an area has been selected and an engineering analysis will be prepared for this area.

A study has been made of an area around Martinsburg, W. Va., for the purpose of selecting a suitable site for an airport to serve that city. This study has been based on interpretation of aerial photography taken in 1938 for the United States Department of Agriculture. The area studied is $8\frac{1}{2}$ miles east to west by $11\frac{1}{2}$ miles north to south with Martinsburg near the center.

Regional Considerations.—

Geography.—Martinsburg, the county seat of Berkeley County, W. Va., is in the east center of the county within the Great Valley that runs northeast through Virginia, West Virginia, and into Pennsylvania. Berkeley County has an area of approximately 192 sq miles and a population of approximately 30,000 (1950). Martinsburg, with approximately 15,000, contains most of this population. The airphotos indicate that the city is largely residential in character and the surrounding area is devoted to agriculture. Orchards cover large areas. Limestone is quarried at many places, probably for use as agricultural lime and cement.

Physiography.—The area of study lies within the Great Valley of the Ridge and Valley Province of the Appalachian Highlands. Fig. 3 illustrates the local physiography. The area is bordered on the northwest by a single sharp-crested mountain ridge, M. This gives way immediately to a broad plain, which varies

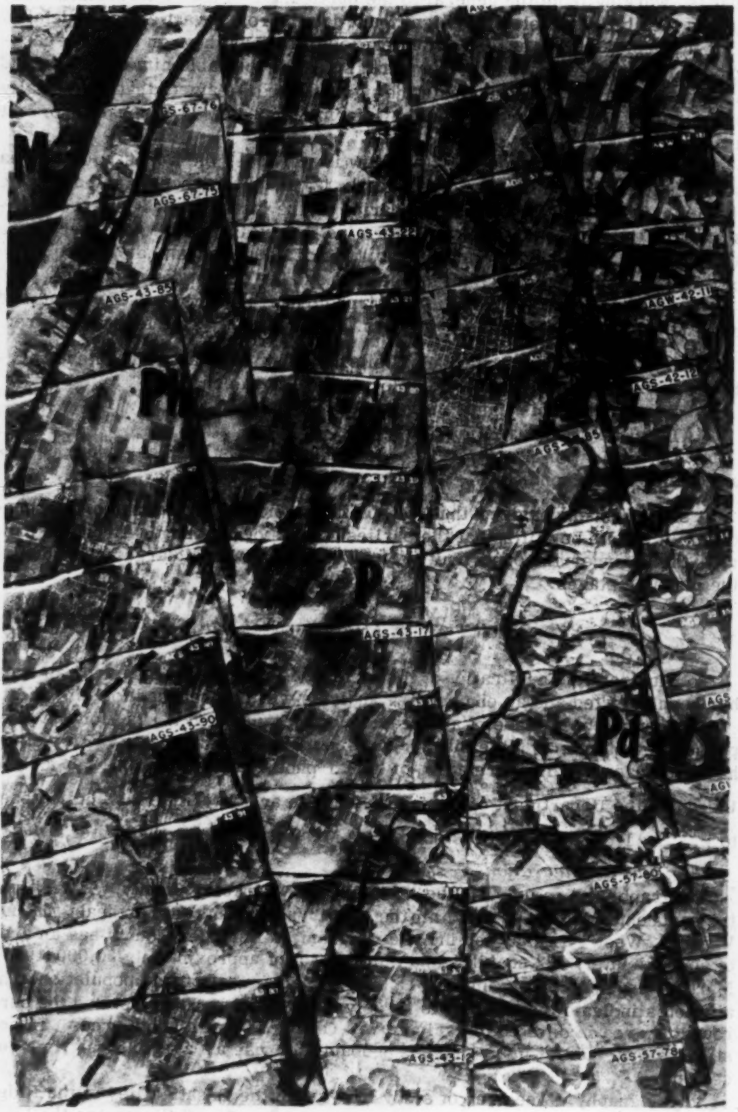
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FIG. 3.—PHYSIOGRAPHIC SUBDIVISIONS OF THE AREA STUDIED

in character from a plain with hills and sharply rolling topography, Ph, to a plain that is nearly level to undulating, P. Throughout this plain a well-defined linear pattern prevails. Bordering the Shenandoah river valley, this plain becomes dissected by tributary erosion. This area, Pd-V, is transected by many deep tributary ravines giving it a hilly topography. The dividing line between these physiographic sections may be drawn by a study of the mosaic of the area.

Geology.—A detailed description of the geology of this area would be very complicated due to the complex geological history of the Ridge and Valley Province. The airphotos provide the answer to local geology. In Fig. 4 Sh indicates shale, Ss denotes sandstone, Ls indicates limestone, and Sh/Ss denotes shale over sandstone. The sharp crest and the steep east-facing slope of the ridge in the northwest corner of the area indicate that this ridge was formed by a resistant sandstone. The bedrock of the plains is composed of steeply tilted limestone and shales. The strong linear pattern of the topography indicates that the bedrock is tilted. That tilted limestone is the predominant bedrock is indicated by the linear pattern of the many sinkholes in Fig. 5. Many of the beds are being quarried for the limestone; a quarry is shown in the upper right corner of Fig. 5. Shale is indicated in areas of more softly rounded topography in the plains section and becomes predominant in the dissected plains section, as evidenced by the absence of linearity and sinkholes. The well-developed surface-drainage pattern is typical of a shale area.

The soils overlying the plains will be predominantly silty clay in texture. In some areas the soil will be very shallow; in others it will probably be 10 ft to 15 ft deep.

Climate.—Heavy woods of the uncultivated areas are an indicator of the subhumid to humid climate of the area.

Local Pattern Features.—The pattern features of each section are discussed to illustrate the method of evaluating and interpreting the soil and terrain conditions.

Mountain Section.—

Landform.—A single sharp-crested ridge trending northeast-southwest. The east-facing slope is a steep escarpment facing onto the plains to the east while the western slope is more gradual with a somewhat rounded appearance. Fig. 6 illustrates the typical landform of this ridge.

Drainage.—Close parallel drainage gullies extend up the western slope. The eastern slope has no drainage system because of its steepness.

Erosional Features.—Heavy vegetation prevents study of the erosion.

Photo Tones.—The vegetation obscures the soil tones except for scattered areas on the east slope where small patches show white to light gray tones.

Vegetation.—Hardwood forest covers both slopes of the ridge, the west slope having a very dense cover. The cover on the east slope is not as dense and shows some lineal stratification.

Cultural Features.—This ridge is too steep for man to impose upon it many cultural features. Widely separated roads cross this ridge with long grades running slightly diagonal to the trend of the ridge.

Analysis.—This ridge standing high above the adjacent plain with its sharp crest and bold, steep eastern slope is typical of a tilted sandstone formation interbedded with other sedimentary rocks. The soft slopes on the west indicate that shale will be found covering the sandstone on this slope.

Plains with Hills Section.

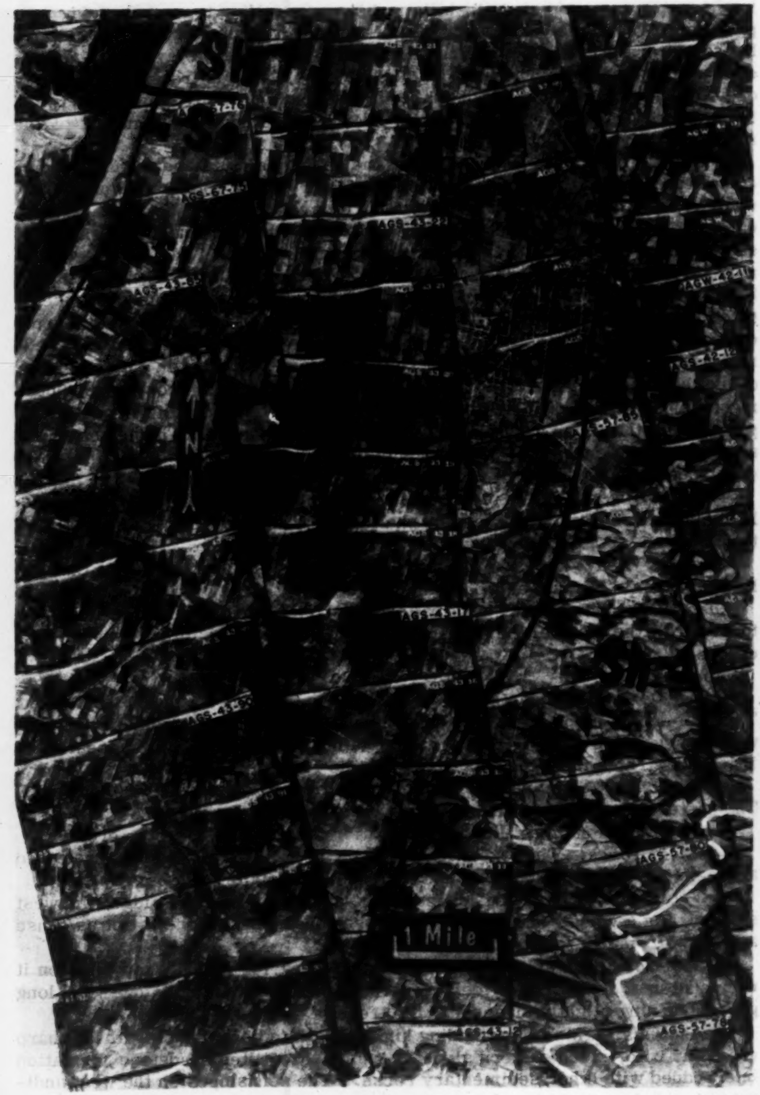
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FIG. 4.—REGIONAL GEOLOGY MAP OF AREA STUDIED

Landform.—This area consists of a plain at the base of the previously cited mountain ridges. This plain has an undulating surface with many low, rounded parallel ridges trending northeast-southwest, which gives the airphoto pattern a definite linear appearance. This linear pattern is modified in areas by the subtle features of sinkholes or solution basins. Fig. 7 is characteristic of the area; note the low parallel hills and the sinkholes in the upper left corner.

Drainage.—No surface drainage system exists. Everywhere the drainage is into small inclosed basins.

Erosional Features.—Gullies are nonexistent. Sheet erosion is undoubtedly predominant.

Photo Tones.—The photo tones vary from white to moderately dark gray. The tone changes are due to both the vegetation and the topography. The steep slopes are usually light with lower areas light to dark gray. Small irregular circular areas of dark gray indicate sinkholes or infiltration basins.

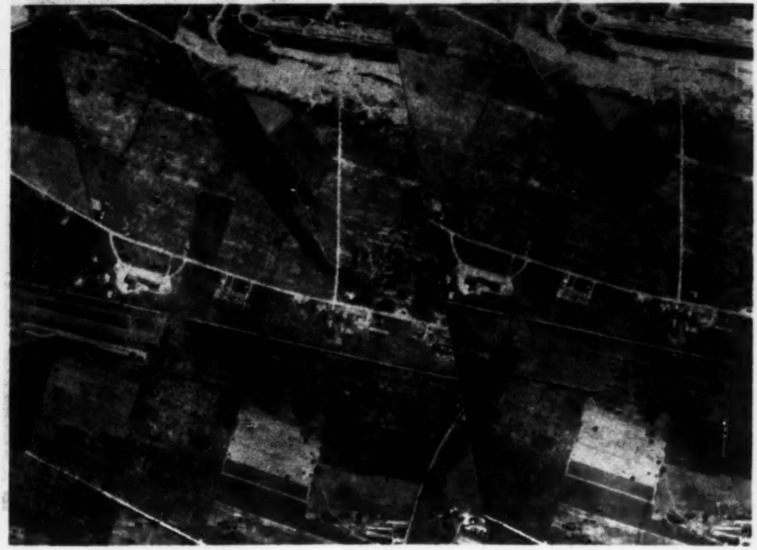

FIG. 5.—STEREOPAIR OF AREA WITHIN THE PLAINS SECTION

Cultural Features.—Except on the steepest slopes the area is intensively farmed. Orchards cover a large portion of the area. Field shapes are usually long and narrow, accenting the linear features of the topography.

Special Features.—The strong linear features and the infiltration basins are a unique combination and each presents strong clues as to the nature of the underlying bedrock.

Analysis.—The linear pattern exhibited by the low parallel ridges indicates that this area is underlain by tilted bedrock. The roundness of the crests and slopes along with the development of small solution basins and shallow sinkholes are characteristic of tilted limestone formations. The lack of development of a surface drainage system bears out this conclusion.

Plains Section.

Landform.—This area consists of level to moderately rolling plains. Low ridges have a parallel trend. Fig. 8 illustrates the terrain features of a nearly level area of the plains. Occasional bare rock outcrops exist at the higher elevations as shown in Fig. 5.

Drainage.—There is no surface drainage system developed except near the eastern edge where the tributaries to the Shenandoah River have reached into the plains. Shallow sinholes exist in many areas.

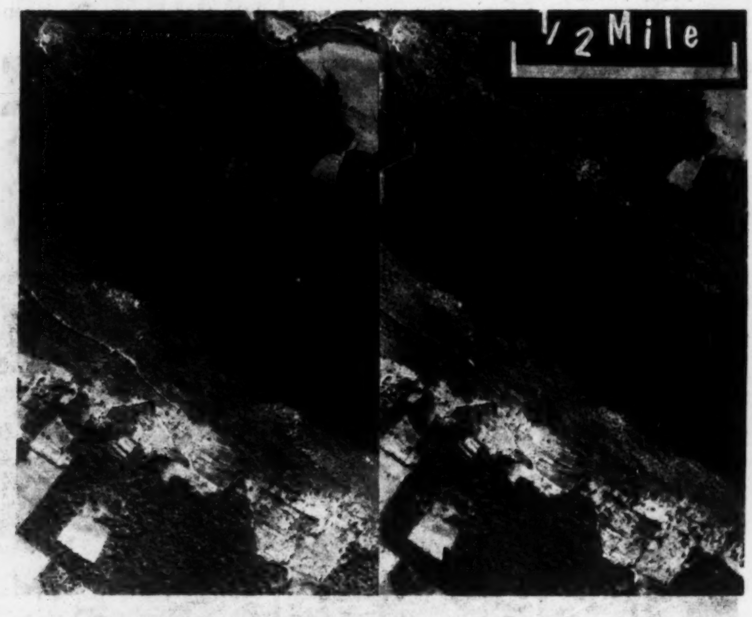

FIG. 6.—AIRPHOTO STEREOPAIR OF SANDSTONE RIDGE

Erosional Features.—Shallow broad gullies are found occasionally. Erosion is predominantly sheet erosion.

Photo Tones.—The photo tones over this plain are most strongly controlled by the crops in the fields, but the tones vary from white to gray. These tones indicate the well drained condition of the soil. Infiltration basins or sinkholes are indicated by dark gray circular areas.

Vegetation.—The present vegetation consists of grain crops and orchards as the area is intensively farmed.

Cultural Features.—The area is intensively farmed. The main roads run parallel to the trend of the shallow ridges. Fields are larger and more rectangular than in the other areas. A northeast-southwest trend, however, is still evident.

Special Features.—Within the area are large quarries that are long, narrow, and deep. These quarries evidently follow a single stratum. These quarries show no refuse pile of stone, indicating that the material is used for agricultural lime or Portland cement.

Analysis.—The low rounded hills with their distinct parallel linear trend and the evidence of sinkholes throughout the area indicate that this area is also underlain by limestone. Quarries in the area also bear out this analysis.

Plains to Valley Section.

Landform.—This area consists of a dissected plain leaving a hilly area bordering the Shenandoah River. The hills mainly trend east-west as they are formed by deep tributary ravines of the river.

FIG. 7.—AIRPHOTO STEREOPAIR OF AREA WITHIN THE PLAINS WITH HILLS SECTION

Drainage.—The predominant drainage consists of deep ravines trending east to the Shenandoah River. A secondary drainage system exists at nearly right angles to this system, giving the overall drainage pattern a trellis appearance as illustrated in Fig. 9.

Erosional Features.—Gullies vary from rounded shapes to deep V-shape indicating differences in soils in the area. Often a gully will broaden out into a sinkhole at its head.

Photo Tones.—Throughout this area photo tones are a darker gray than in other areas.

Vegetation.—The area is farmed with fields of grain crops except in the more extremely dissected areas where hardwood forests stand.

Cultural Features.—The area is farmed in small irregular fields. Roads are strongly controlled by the topography and follow the ridge tops.

Special Features.—Within this area there is a brick plant and a quarry, indicating the nature of the material.

Analysis.—This section is composed of a plains dissected to the stage that it is a region of soft rounded hills. A well-developed trellis drainage system indicates that internal drainage is not the main source of drainage. The underlying rock is more impervious than the other areas. This all indicates that the area is underlain by interbedded sedimentaries in which shale is the predominant member.

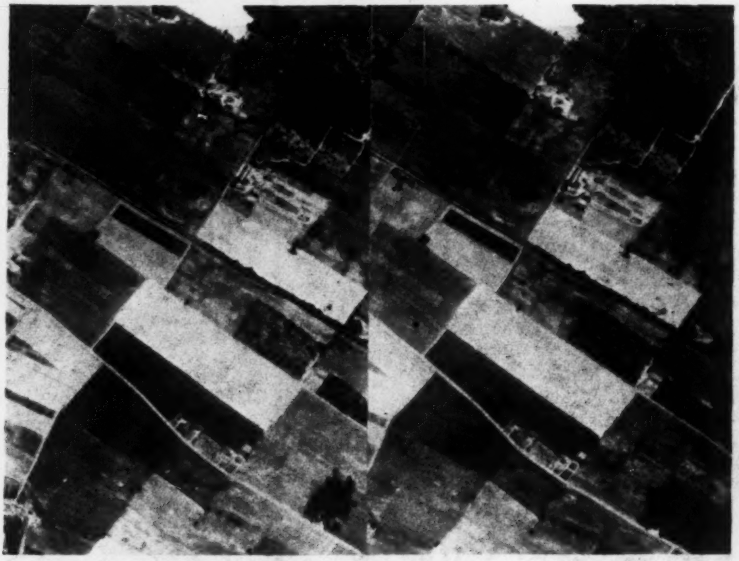

FIG. 8.—AIRPHOTO STEREOPAIR OF NEARLY LEVEL AREA OF PLAINS SECTION

Engineering Analysis and Appraisal.—Throughout the area the topography presents a strong parallel linear trend. The mountains, the low ridges of the plains area all trend northeast-southwest. Throughout much of the area the topography has too much relief to provide an adequate site for an airport. Any airport must be located far enough east of the high ridge so that the ridge does not become an approach obstacle. Only in the plains section is the terrain level enough for location of an airport.

The soils of the plains are residual soils developed from the underlying limestone. They are predominantly a plastic clay or silty clay with good internal drainage in the undisturbed state. These soils will be impervious upon being remolded during construction. The depth of these soils will vary. Many areas where bedrock outcrops or is near the surface are found in studying the photos. In other places the soils will be 10 ft to 15 ft deep.

The soils of the dissected area near the river are probably residual soils developed over interbedded limestone and shale. These soils are not well drained internally.

In the undisturbed state, the soils of the plains are well drained internally. The sinkholes may be a problem for some construction. Drainage ditches should be provided to remove water from the edges of any runways as these soils will no longer have good internal drainage and are apt to swell and lose strength if the surface water is not drained away from the area.

Embankments for runways will have to be constructed from the local residual soils. Base course and paving materials can be obtained from the limestone quarries in the area.

APPLICATION OF INFORMATION TO AIRFIELD SITE SELECTION

Site Selection Criteria.—The following criteria were established before the airphoto study of sites was begun. These do not, of course, establish the need or the ability of the community to support an airport.

1. Close proximity to the city served.
2. Moderately level terrain to reduce cost of construction.
3. Low-cost land.
4. No nearby approach obstacles.
5. Runways 5,000 ft long.
6. Best possible soil and drainage conditions.

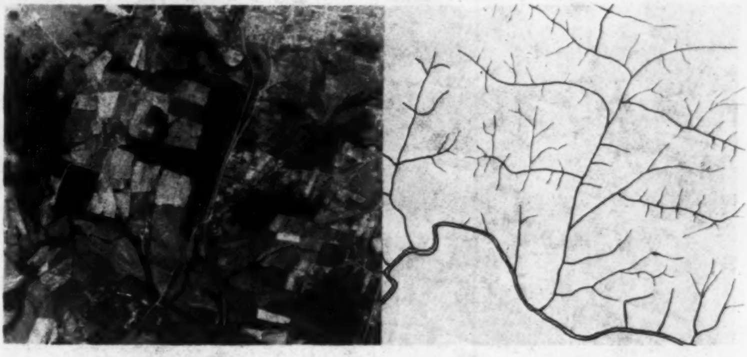

FIG. 9.—AIRPHOTO AND DRAINAGE MAP

Site Selection.—From the study of the airphotos three sites were selected for preliminary study. The locations of these three sites are shown in Fig. 10.

Site 1.—This site is situated about 3 miles north of Martinsburg just west of U. S. Highway 11 and has good access by U. S. 11. The site is bounded by U. S. 11 on the east, State Highway 9 along the southwest, a railroad track running northwest-southeast along the north side and a railroad track running north-south along the west side.

The terrain is moderately to sharply rolling with local relief being approximately 20 ft. This terrain would require considerable cut and fill for runways.

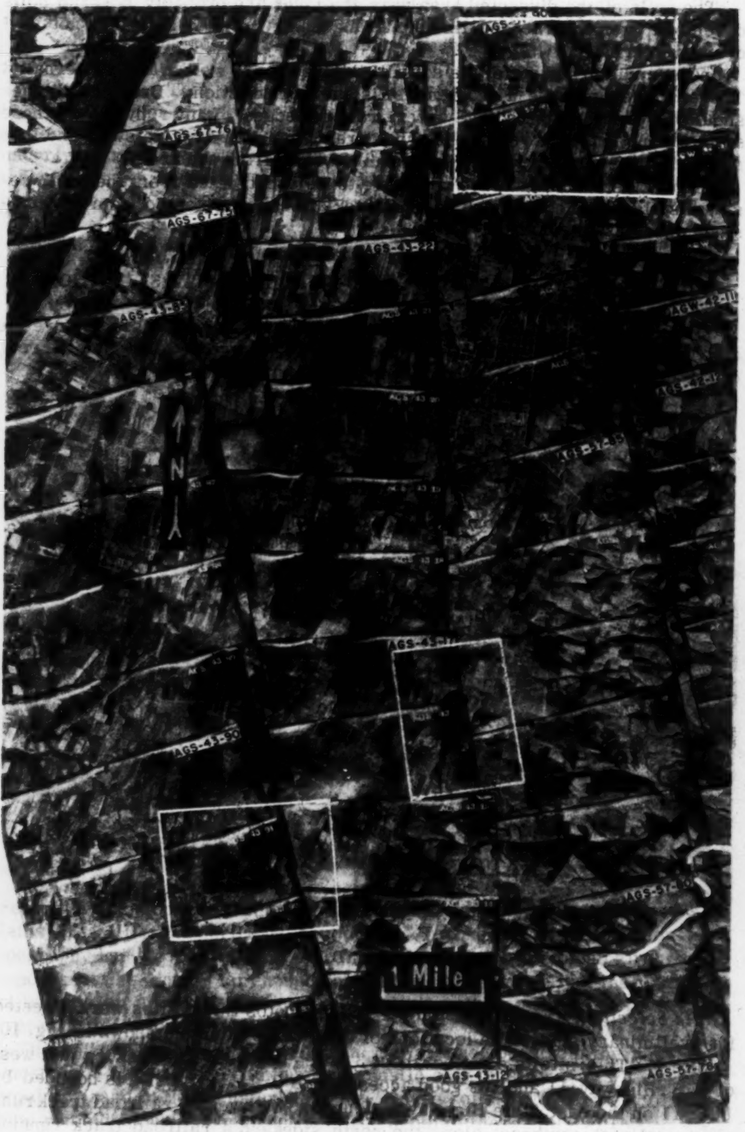
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FIG. 10.—SITES STUDIED FOR POSSIBLE LOCATION OF AIRPORT

The soils of this site are residual soils developed from limestone bedrock. The airphoto indicates that the bedrock is close to the surface in many of the high areas, which means many rock cuts during construction (Fig. 11).

The site is limited by being bounded on all sides by the highways and the railroad, and has another major drawback in its close proximity to the high mountain ridge.

Site 2.—Site 2 (Fig. 12) is located approximately 3 miles south of the city just east of U. S. 11. The east-west width of the site is limited by the highway and the dissected hilly terrain to the east. The site has good access from the city by U. S. 11.

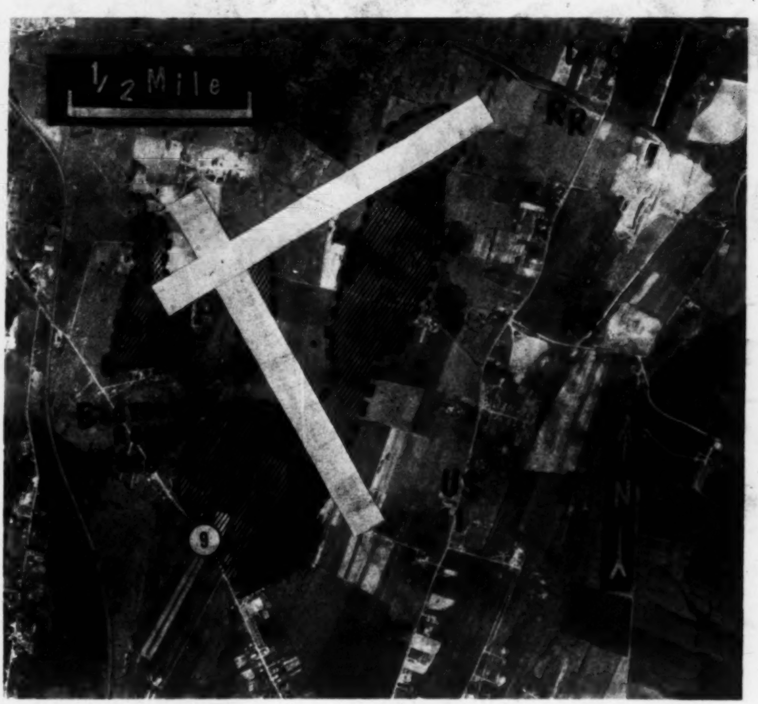

FIG. 11.—SITE MAP FOR SITE 1

The terrain is rolling and will require some deep cuts for the runways. There is no evidence, though, that bedrock is near the surface.

The soils in this site vary from plastic silty clay to clay residual soils developed over limestone to residual silty clay soils developed over shales and interbedded limestone. The soils developed over shale are predominant and cover the east two-thirds of the site. These soils will provide poor subgrades which will require thick base and sub-base courses.

A drainage ditch should be provided along the south edge of the southeast leg of the runways as the runway transects a natural drainage course. A drainage map of the site is shown in Fig. 13.

A low ridge transects the ends of both runways and will require excavating to establish runway grades. The soil, however, appears to be deep in this area and rock cut will probably not be extensive.

Site 3.—Site 3 is located approximately 6 miles southwest of the city just west of U. S. 11, which would provide good access (Fig. 14).

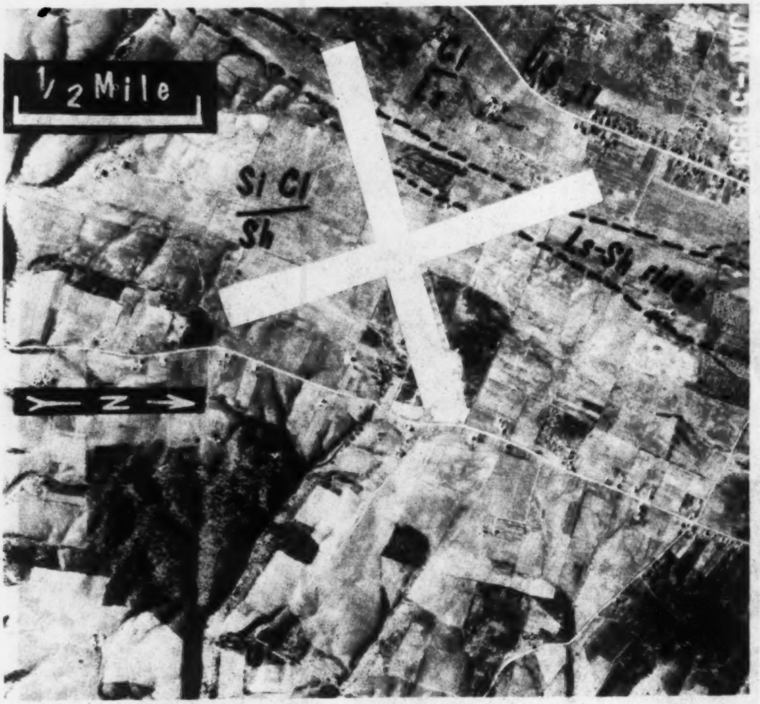

FIG. 12.—SITE MAP FOR PROPOSED SITE 2

Over the south one-half of the site the terrain is only slightly rolling. In the north half the site has considerable more relief with low hills rising 30 ft or more. Much of the local relief is created by the sinkholes and solution basins.

The soils in the area are residual soils developed from the underlying limestone. The soils are very shallow over a large part of the area and hard rock cuts would be required to provide a runway. There are many sinkholes in the area which would provide another problem. These may require grouting or bridging wherever they would occur under a runway.

The present use of the area consists of farming, with three orchards in the area required for runways.

Comparative Analysis.—Before the final selection, field exploration should be made at each site and land appraisals made to provide an adequate cost estimate of each site.

A comparison of the sites from the airphoto study indicates that Site 2 is probably the best of the three sites. Sites 1 and 3 have rock near the surface and rock cuts would be required. The undulating topography of Site 1 would require considerable earth movement to establish runway grades. Over most of Site 3 the topography is quite level but the low hills in the north part will require deep rock cuts. The sinkholes found over much of Site 3 would add a considerable cost to construction. The topography at Site 2 controls the directions of the runways and cuts will have to be made to establish a runway grade. There is, however, evidence to indicate that the soils are deeper in this area and rock cuts would not be so extensive. Sites 1 and 3 are much nearer the mountain ridge to the west, which may provide an approach obstacle.

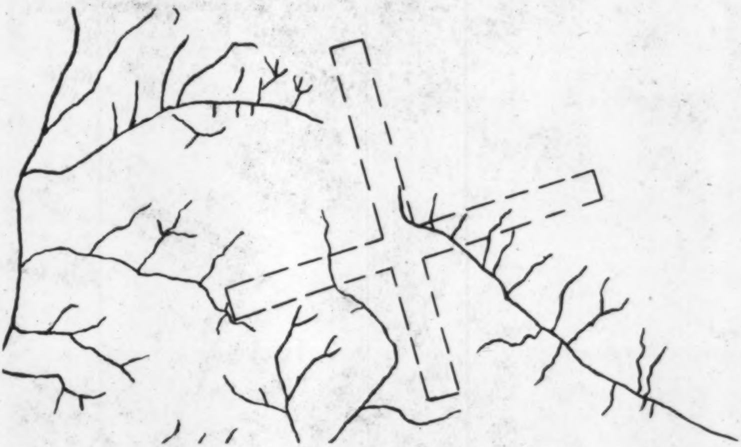

FIG. 13.—DRAINAGE MAP OF SITE 2

All three sites have the same general type of soil, but as stated the soil is deeper at Site 2 than at the other sites.

Access is good to all sites, with U. S. 11 running near each one.

If land costs at all sites are nearly equal, Site 2 because of the lack of extensive rock cuts, will probably provide the most economical site that will serve the needs of the community.

SUMMARY

This report illustrates the use of airphoto interpretation in airport-site selection. The information used to select the sites could be obtained in many ways. The use of topographic maps, geology, and field data would develop the required information. Airphoto interpretation gives the engineer a perspective that none of these other items can. He is able to make a comprehensive evaluation of all the environmental factors that enter into the analysis.

It is not suggested that the airphotos will provide all the answers, but then neither does a soil auger, or a geology report. The airphoto is a tool that should be used with all the other means of obtaining information. It should not supplant field investigation. Instead it should be used to plan and as an aid to field explorations. Its use should start at the first planning stage and continue hand in hand with all other planning and investigations.

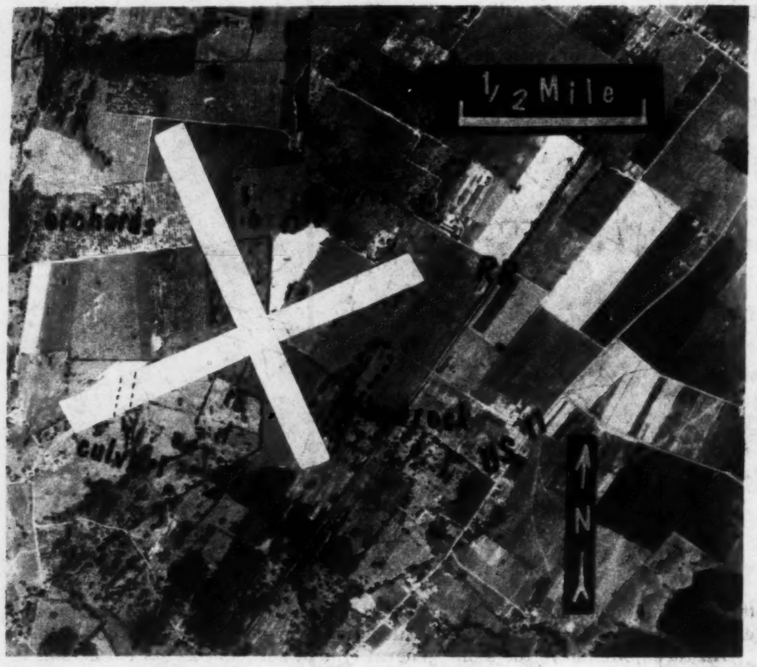

FIG. 14.—SITE MAP FOR SITE 3

APPENDIX.—REFERENCES

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 3165

CORRELATION OF VEHICLE DESIGN AND HIGHWAY DESIGN

By Richard A. Haber,[1] F. ASCE, and David K. Witheford,[2] M. ASCE

SYNOPSIS

Past and present developments in the highway field indicate that insufficient coordination of vehicle design and highway design is placing the highway program in a precarious position. Current highway expenditures and those contemplated for the future represent an investment that must be protected. The situation calls for a unified research approach supported by industry and all interested groups and agencies.

INTRODUCTION

Denouncing the auto industry for the production of long, low, flashy, finny monsters of questionable design has become a favorite after dinner sport. At the same time many people revel in their new prestige as owners of "nice little foreign cars which invoke feelings of owning a finely machined piece of Old World Craftsmanship." Is this thought pattern a trend which will be met by the growth of American-made, compact cars? Have economies in cost and operation become essential to design criteria? Or is the "compact car" considered by manufacturers a backward or side step in the competitive race to a new sized, "automated" family vehicle of the future? Some hints concerning the vehicles of the future have been offered. The development of controls to be used in them is in process. It is not strange that the entire subject lends itself to conversational speculation, by highway engineers as well as the public.

Note.—Published essentially as printed here, in June, 1960, in the Journal of the Highway Division, as Proceedings Paper 2529. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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No less popular is the perpetual subject of taxes, and whether the American taxpayer is receiving services commensurate with the funds he is providing. Signs of this thinking are obviously behind the current investigations of the highway program initiated by the White House and Congress. The present delays to highway progress are perhaps due in part to a lack of official confidence in our ability to plan properly, build well, and spend wisely the public funds that have been provided.

Highway engineers and builders do not have this lack of confidence and have expressed confidence in their ability to do the job. They should not rest on this belief and fail to guard against these factors that lead to charges of inefficiency and waste in the use of public funds.

Possibly the most important factor contributing to imagined and real waste of highway funds is untimely obsolescence. All highway engineers have seen examples of highways costing millions of dollars rendered inadequate by unpreventable right-of-way development, necessary traffic controls, or traffic volumes rising beyond all expectations. Highway engineers have developed measures to apply against these defects, such as access control, grade separations, and more refined traffic forecasting techniques. There is, however, another less obvious and more insidious cause of highway obsolescence; one over which the highway industry has little influence. It stems directly from the characteristics of the vehicles using the nation's highways.

The objective here is to outline what has happened to highways because of changing vehicle characteristics. The purpose is to urge the development of a unified approach by the motor vehicle designers and the highway engineers to their inseparable problems. The need for such correlation has been present for the past 50 yr. This need has been met partially, and with reasonable success. If it had not, today's highways would be even more inadequate. But, with problems increasing because of the massive highway program, efforts must also increase to achieve optimum highway value.

It is vital that a greater measure of communication between vehicle designers and highway designers be put into action, without sacrificing the creativity of either group by the establishment of unnecessary controls.

Why is this need so acute at the present time? The answer is obvious. The trend to larger and heavier commercial vehicles is more apparent than ever. In addition, expenditures for highway improvements are at their highest level in history. That they will continue to increase in the years ahead is generally accepted. The challenge is in the present and future but the past contains the lessons. It is worthwhile therefore reviewing what has happened in the past. The year 1927 is a good place to begin.

BASIC PHILOSOPHIES

By 1927, both the automobile and its routes of operation had been on the public scene long enough for philosophies to have been crystallized by the respective interested groups of vehicle manufacturers, highway builders, and the public. Examination of these philosophies reveals the fundamental reasons for the divergence of viewpoint that existed then even as today.

The basic principles of free enterprise and competition govern the manufacturer's outlook. His tools and plants are built with private capital which demands a maximum cash-in-hand return on the investment. His products must compete with those of other manufacturers for the private "convenience capital" of the consumer. In a larger sense, the vehicle manufacturer is also competing with other forms of transportation in an effort to broaden his markets. Consequently, frequent, detailed, design changes have become necessary to meet competition. The result is that each end product has a relatively short-lived appeal and is annually replaced by something hoped to have an even greater public appeal. This accepted manner to provide the maximum return on capital investment, has been labeled "planned obsolescence."

On the other hand, the highway administrator is governed by legislative bodies which establish funds and procedures for his operations. For example, the law establishing the Delaware State Highway Department in 1917 required that highways should be designed for a 40-yr-life. In any case, the fact that the highway engineer draws on "involuntary receipts" (commonly called taxes) for capital expenditures requires him to design for long use, with economy in construction and maintenance as co-equal principles. The capital investments are non-recoverable, except in terms of the service provided users and non-users. These considerations are a major influence toward economical design.

The road-user provides funds through taxation for the highway system. He also provides income for the manufacturers by purchasing their products. Although the individual may feel that he is being pushed and pulled continually by both groups, the fact remains that he is the one who establishes the directions taken by the other two.

Inevitably and understandably, the road user dislikes being taxed for services if dubious about the measure of service provided. He demands that his tax dollars yield maximum benefits. This sometimes leads to contradictions, depending on whether he is sitting behind the wheel or behind the chairman's desk at a civic association meeting. When making a voluntary expenditure, he wants the most for his money. In the case of passenger vehicles, this has been interpreted to mean that cars must become longer, wider, more powerful, more comfortable, and more "stylish" (which has been achieved partly by lowering height to give apparent greater length).

In commercial highway transportation, economy of operation is critical. This has been obtained in part by making each unit capable of hauling heavier loads at greater speeds. Although the operator realizes this economy, he also necessitates a relatively greater use of tax dollars to accommodate such units.

Admittedly, this has been an over-simplified statement of controlling influences, but is, nonetheless, relevant to the following outline of vehicle and highway design development.

VEHICLE DESIGN DEVELOPMENTS

The technological skills and creative imagination of the American automotive industry cannot be doubted. Its ability to produce vehicles of high quality in vast numbers at low cost has earned worldwide respect and envy. But the pace of development in this industry has caused many of the problems in the highway profession. Some of the changes in design are shown in the accompanying figures and tabulations. An expression of debt must be included here to the researchers in both industry and public service who have compiled these figures.

In connection with passenger vehicle characteristics, it can be seen from Fig. 1 (a) the average lengths of 4-door sedans have increased since 1927 by 46 in. In the so-called, "low priced three," the length change has been 60 in. Average widths during the same period, shown in 1 (b) have increased slightly over 11 in. to the present 78 in. The effective width change for parking purposes, is actually greater since the doors have been moved outward to the fender line. As shown in Fig. 2. The only dimension which has decreased has been height. It has dropped from an average of 73.7 in. to 57.4 in., a reduction of 16.3 in. Data on angles relating to the underclearance of passenger vehicles are illustrated in Fig. 3 and show the trends since 1948. The angle of approach which averages 21°, with a minimum sample at approximately 14°, has not changed. However, the mean angle of departure has decreased from 16° to a fairly stabilized level of 12° to 13°. The minimum angle of departure was 9° in 1959. The ramp breakover angle has decreased from an average 16° to approximately 11°, with a 1959 minimum of 7 1/2° on one American-made sports car. There has been little change in ground clearance, averaging just under 6 in. for all models, with a minimum of 4 in. The figures relating to underclearance do not take into account conditions of "jounce" or spring compression, that further reduce the angles or clearances in dynamic situations. Nevertheless these figures are meaningful to those concerned with the abrupt grade changes that occur at ramps and driveways.

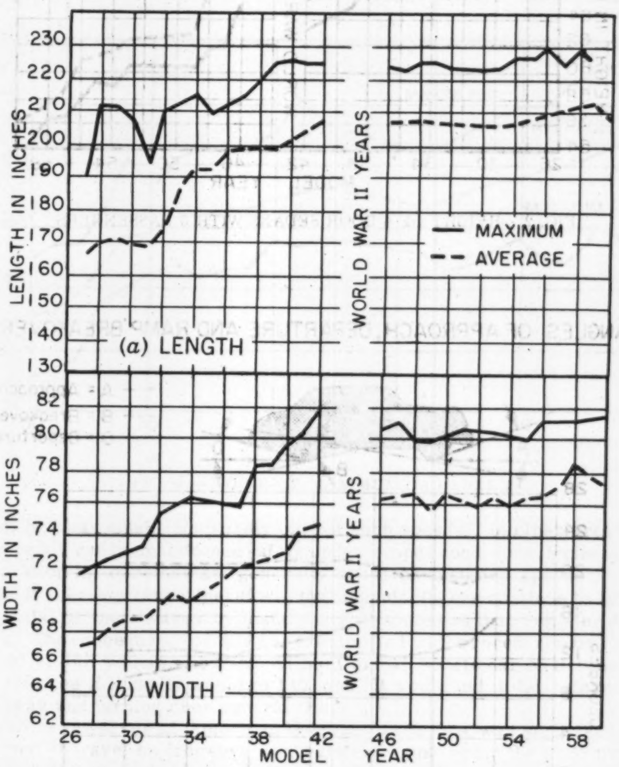

FIG. 1.—LENGTH AND WIDTH OF 5 TO 6 PASSENGER, 4 DOOR SEDANS

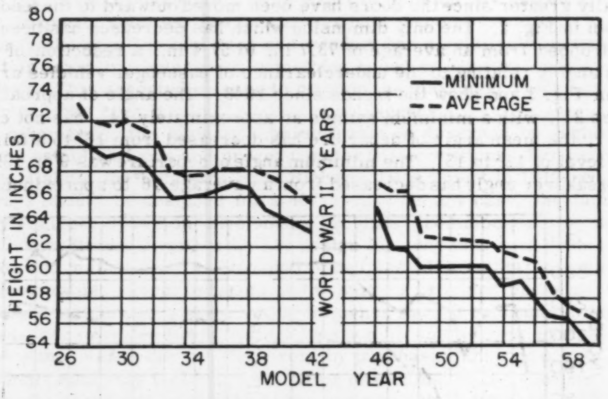

FIG. 2.—HEIGHT OF 4 DOOR SEDANS WITH 5 PASSENGERS

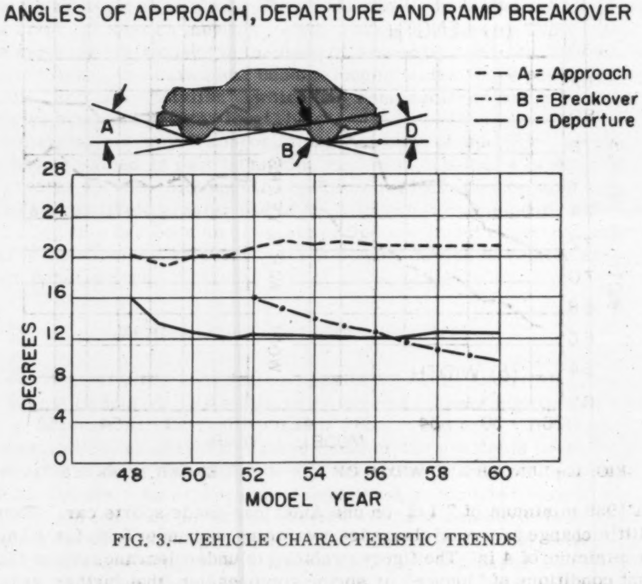

FIG. 3.—VEHICLE CHARACTERISTIC TRENDS

The passenger vehicle dimension of greatest concern to the highway engineer and which has decreased most is vehicle height. As shown by Clyde E. Lee[3]. The average driver eye height had dropped to less than 4 ft. in 1957 and later model cars, and an estimated height of 3.95 ft must be used to accommodate 85% of the drivers on the highways today. This factor, combined with the speed increases noted in Fig. 4, illustrating maximum speed capabilities, results in considerable modification to vertical curve design-as will be shown later.

Speed is an absorbing topic in itself. From 1930 to 1955, the maximum speed potential of the average American built car rose from 66 mph to 97 mph. In 1955, the lowest figure for maximum speed was approximately 80 mph. This speed potential is seldom utilized by the average driver. On the East Coast, at least, highway conditions rarely permit such speeds. In other areas where higher speeds can be maintained safely under proper conditions, highway design and operation is, nonetheless, generally based on assumed speeds of 70 mph or less. It seems strange, therefore, that manufacturers continue to advertise the potential of their wares by installing speedometers that register 120 mph or more. The consequence of this was exemplified by the recent report concerning the 60 yr-old woman who tried out her German sports car on Britain's first freeway, to see if it really would go 140 mph. It would and it did, before leaving the highway and turning over several times.

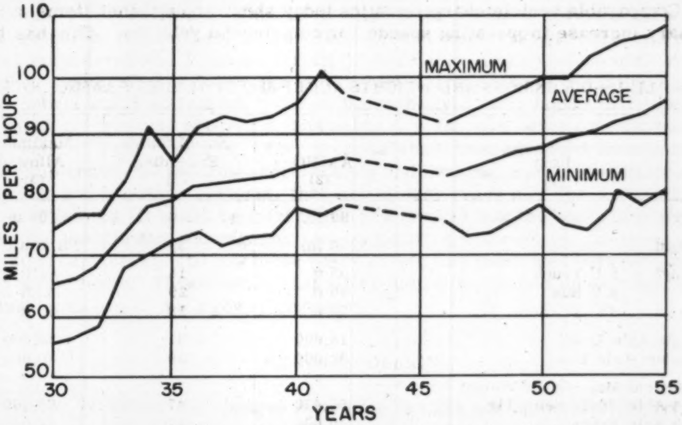

FIG. 4.—TREND OF MAXIMUM CAR SPEED

A British observer of American freeway operations was surprised to find that "speed of travel on freeways is consistently just under the maximum permitted and the remarkable lane discipline". These conditions, observed on Detroit freeways, probably resulted from the volume of traffic, as well as reflecting the effectiveness of enforcement practices and driver education. They possibly also reflect, under certain circumstances, a public awareness of the need to resist the inducements to speed offered by the vehicle manufacturers.

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[3] "Driver Eye Height and Related Design Features," by Clyde E. Lee, The Univ. of Tex., Bur. of Engrg. Research, Austin, Tex., 1959.

The claim has been made that the changes in size and speed potentials of passenger vehicles have been dictated by customer demand. Certainly these demands, if they exist, are based on the occasional maximum need of the user for passenger space, storage capacity, and speed of travel. The ingenuity of the industry in meeting these demands is remarkable. The sense of responsibility toward the obvious effects on highway systems and traffic operations has been somewhat less than remarkable.

Much more significant to the highway engineer is the change in commercial vehicles. In 1927, highway design standards were based on design speeds of 35 mph; most bridges were designed for H-15 loadings; bus widths were 8 ft; truck widths were 7-7 1/2 ft; and maximum gross vehicle weights were approximately 30,000 lb.

Comparable vehicle characteristics today show, among other items, a necessary increase in operating speeds for commercial vehicles. This has been accompanied, however, by official distrust of other operating characteristics, that has resulted frequently in lower speed limits than those that apply to cars on the same highways.

TABLE 1.—DIMENSIONS AND WEIGHTS ACCEPTED IN CURRENT AASHO POLICY

Item (1)	AASHO (2)	No. of States Exceeding (3)	Maximum Allowed (4)
Width	96 in.	3	108 in.
Height	12-6 in.	30	No limit
Length - S U Truck	35 ft	18	42 ft
S U Bus	40 ft	29	55 ft
Tr. Tr. Semi Tlr.	50 ft	18	65 ft
Single Axle Load	18,000	31	24,000
Tandem Axle Load	32,000	29	40,000
Practical Max. Gross Weight			
4 Axle Tr Tr Semi Tlr	55,470	47	66,400
3 Axle Truck	40,000	27	52,000

Table 1 indicates dimensions and weights accepted in current AASHO policy, with pertinent exceptions.

The major changes since 1927 have obviously been in weights and lengths of vehicles, while widths and heights have remained relatively unchanged. It would appear that the truck manufacturers have been forced to follow design policies similar to those of railroads, whose fixed gauge, track spacing, and clearances in terminal facilities have required them to improve equipment and reduce operating costs within those limits. Since highways are, more or less, fixed assets similar to railroad trackage, highway design, within certain limitations, would certainly seem to be appropriate.

That this is not the case can be seen from recent developments, that indicate the continued trend to larger size and loadings. An example can be drawn from the New York Thruway, which now permits the operation of 100 ft long combinations with gross weights up to 130,000 lb. Motivated by a logical desire for increased freight hauling efficiency, both industry and users are pressing for greater vehicle widths and lengths exceeding current AASHO policy (although still below some States' tolerances). Removal of restrictions on the number of units in combination, and increases in allowable single and tandem axle loadings are also desired. A composite picture of the suggested freight vehicle of the near future suggests a width of 9 ft, height of 13 ft 6 in., length up to 100 ft, and gross vehicle weight close to 200,000 lb. There are apparently no vehicle design problems that would prevent such development.

It has been noted that no major height increase is anticipated. Yet, the United States Department of Defense has requested a 17 ft clearance for structures on the interstate system. It is believed that not only will this be provided, but that it will result ultimately in requirements of greater clearances on other systems for both public and private vehicles.

Some reference must be made to other future vehicle developments. The air-supported vehicle that travels without ground contact has proved to be a practical consideration. It would certainly solve axle loading problems. An inherent requirement in its design is a smooth surface free of the fallen objects and debris which so frequently litter the highways. It is hoped that solutions to parking, driveway problems, and urban driving, are being considered concurrently with such designs. Other proposals have been advanced, incorporating fingertip control of steering, acceleration and braking, coupled with automatic warning and guidance systems, that are almost ready now for application to rural freeways. It would be interesting to know (a) how the costs of equipping both the vehicle and the highway can be justified in terms of safety or more efficient service, and (b) how these costs should be apportioned between private and public funds. These items, that affect future design and investment, are a natural concern of the highway engineer.

HIGHWAY DESIGN

Having examined the changes in vehicle designs during the past 30 yr, it is fitting to review the changes in highway design that have occurred during the same period. In their way they have been just as drastic as those in the vehicles.

Highway design is influenced basically by four factors, the volume, speed, size, land weight of traffic. Over the past 40 yr there has been a shift in emphasis from the static design aspects (cross-section, profile, sub-grade, and drainage), that were important in obtaining economical construction and life, to the dynamic aspects of geometric design needed for high operating speeds and smooth flow of traffic. Table 2 indicates design standards for two-lane primary rural highways for both 1927 and the present day. It reflects the shift from the static to the dynamic, but more importantly it reflects the changes in highway design which have been necessitated by changes in vehicle characteristics. It will be noted that design speeds have doubled and thus increased the requirements for pavement and roadway widths, bridge widths, and horizontal alignment.

The change in pavement design is well illustrated by Table 3, which lists pavement design trends from Delaware's road life files. The decade-by-decade growth in pavement widths and thickness is clearly evident. Computation of the pavement quantity required to provide a 25-ft slab 10 in. thick compared with that to provide an 18-ft slab 7 in. thick shows that nearly twice as much pavement is required per linear foot. The contrast is even more marked by comparing Figs. 5 and 6. Fig. 5 shows a 16 ft secondary road with a concrete pavement constructed in 1926 (photo taken in 1934 after adding wearing course).

TABLE 2.—DESIGN STANDARDS TWO-LANE PRIMARY RURAL HIGHWAYS

Items (1)	1927 (2)	Current (3)
Design Speed, in miles per hour	35	70
Right of Way, in feet	80	100-120
Bridge Loads	H-15	H20-S16
Bridge Roadway, Width, in feet	24-30	44
Grades, in %	5-6	3-5
Horizontal Curve Radius, in feet	600	1500-1800
Roadway Width, in feet	32-40	44
Pavement Width, in feet	18-20	24
Sight Distance, in feet	500 (Passing)	600 (Stopping)
Eye Height, in feet	5.5	4.0 (Desirable)
Vertical Clearance, in feet	14	14

TABLE 3.—RIGID PAVEMENT DESIGN TRENDS TWO-LANE RURAL HIGHWAYS

Decade (1)	System (2)	Pavement Width, in feet (3)	Pavement Thickness, in inches (4)
1920's	Primary	18	7
	Secondary	16	7
1930's	Primary	20-22	8
	Secondary	16	7
1940's	Primary	22	8
	Secondary	18	7
1950's	Interstate	25[a]	10
	Primary	24	9
	Secondary	24	8

[a] One-Half Divided Highway.

Fig. 6 is a recent photograph of a major primary route, with six 12-ft running lanes, adequate median, and collector-distributor roads, each with two 12-ft lanes.

The need for adequate shoulder width to provide lateral clearance and emergency parking areas has been generally realized through the years. The serious

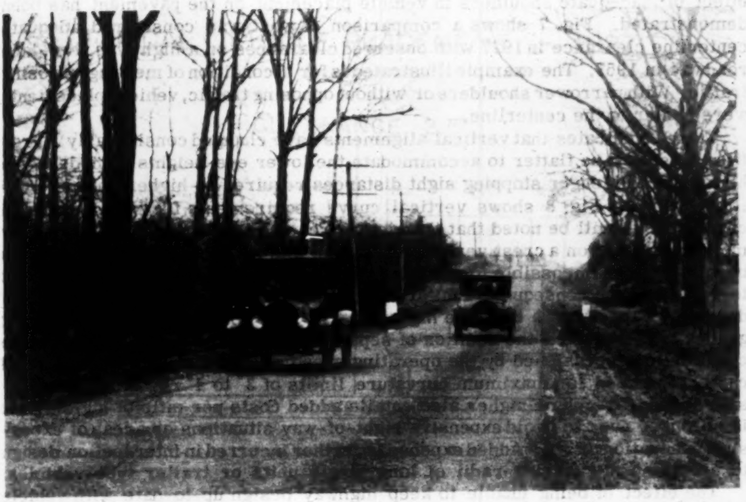


FIG. 5.—ADEQUATE SECONDARY ROAD OF THE 1920's

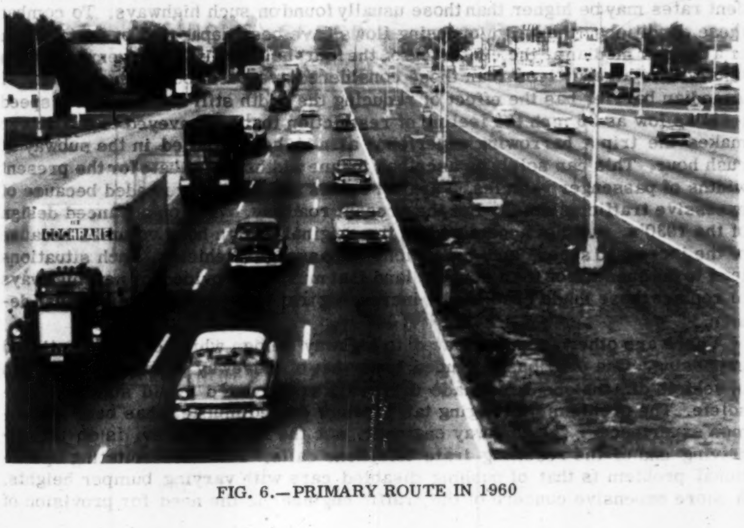


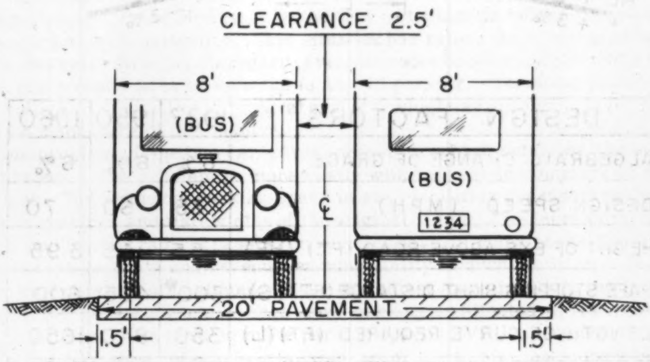
FIG. 6.—PRIMARY ROUTE IN 1960

effect of inadequate shoulders in vehicle placement on the pavement has been demonstrated. Fig. 7 shows a comparison of what was considered adequate centerline clearance in 1927 with observed clearances on a high type, two-lane roadway in 1957. The example illustrated is for a condition of meeting opposing traffic. With narrower shoulders or without opposing traffic, vehicle placements were closer to the centerline.

Table 2 indicates that vertical alignments have changed considerably. Profiles must now be flatter to accommodate the lower eye-heights of modern vehicles and the longer stopping sight distances required by higher speeds. The illustration in Fig. 8 shows vertical curve requirements for various design conditions. It will be noted that the current requirement calls for considerably more excavation on a crest vertical curve than was necessary 30 yr ago. It has become almost impossible to provide passing sight distance on other than flat terrain, and the consequent limitation in passing opportunities effectively reduces the capacity of a two lane highway. Two lane roads with continous long grades have required the addition of separate climbing lanes to overcome the capacity loss occasioned by the operating speed differential between truck and cars. The need for maximum curvature limits of 3° to 4° resulting from design speeds 70 mph or higher also entails added costs per mile of highway. It is no longer easy to avoid expensive right-of-way situations or areas of excessive excavation or fill. Added expense is further incurred in intersection design to accommodate turning radii of long single units or trailer type vehicles.

The effect of being unable to keep highway design up-to-date with vehicle design is obvious from the following example. The Cross County Parkway north of New York City was adequately designed for the vehicle-operating characteristics of the times. Its operational characteristics today have not been affected by either right-of-way encroachments or the introduction of at grade intersections. Traffic volumes have, no doubt, increased beyond capacity and accident rates may be higher than those usually found on such highways. To combat these conditions, the heavy opposing flows have been separated by installation of a barrier median at the centerline of the four-lane, undivided pavement. With lane widths already less than those considered adequate today, the addition of a median barrier has the effect of reducing the width still further. At a speed level as low as 35 mph the feeling of restriction that is conveyed to the driver makes the trip a harrowing experience akin to being jostled in the subway at rush hour. This can solely be ascribed to lane widths inadequate for the present widths of passenger vehicles. Parallel routes are evidently needed because of excessive traffic volumes. But the present roadway, based on advanced design of the 1930's, will not accommodate the original design hour volumes because of the increased speeds and dimensions of passenger vehicles. Such situations lead to the question of the amount of land that must be provided for new highways to replace those made obsolete by increasing traffic or inadequate original design.

There are other aspects related to highway design which can be mentioned in passing. One previously alluded to is that of driveway problems. Changes in vehicle dimensions have made driveways constructed to old standards obsolete. The problem of dragging tail pipes of dented mufflers has been a common experience. The highway engineer, not the manufacturer, is on the receiving end of the resulting irate telephone calls. Another irritating operational problem is that of pushing disabled cars with varying bumper heights. A more expensive concern of the traffic engineer is the need for provision of (*a*) TYPICAL VEHICLE PLACEMENT - 1927


CLEARANCE 2.5'
8'
8'
(BUS)
(BUS)
1234
℄
20' PAVEMENT
1.5'
1.5'

(*b*) TYPICAL VEHICLE PLACEMENT - 1957

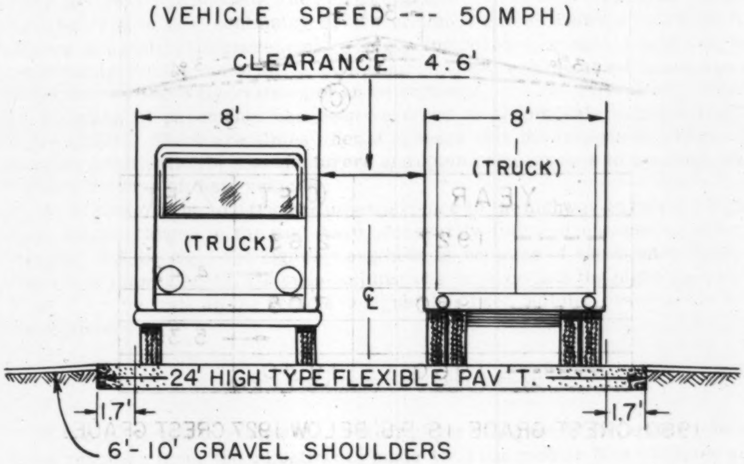
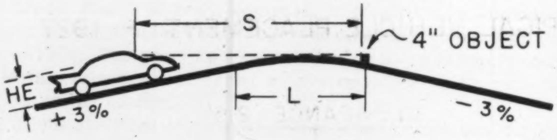
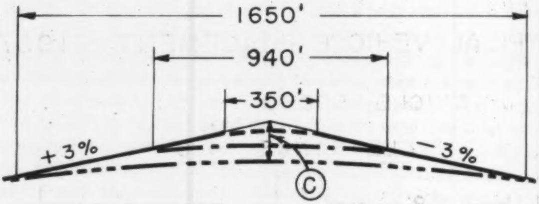

(VEHICLE SPEED --- 50 MPH)
CLEARANCE 4.6'
8'
8'
(TRUCK)
(TRUCK)
℄
24' HIGH TYPE FLEXIBLE PAV'T.
1.7'
1.7'
6'-10' GRAVEL SHOULDERS

FIG. 7.—TYPICAL VEHICLE PLACEMENTS



DESIGN FACTORS	1927	1950	1960
ALGEBRAIC CHANGE OF GRADE	6 %	6 %	6 %
DESIGN SPEED (MPH)	35	60	70
HEIGHT OF EYE ABOVE ROAD (FT.) (HE)	5.5	4.5	3.95
SAFE STOPPING SIGHT DISTANCE (FT.) (S)	300*	475	600
LENGTH OF CURVE REQUIRED (FT.) (L)	350	940	1650

**approximate*



YEAR	© FT.	△ FT.
---- 1927	2.63	
		← 4.42
—-— 1950	7.05	
		← 5.35
—---— 1960	12.40	

1960 CREST GRADE IS 9.8' BELOW 1927 CREST GRADE.

FIG. 8.—VERTICAL CURVE DESIGN 1927 TO 1960

traffic signals over each lane on multi-lane facilities because vision is obscured by the heights and widths of trucks. Another item of concern to the highway or vehicle commission administrator is that of design modifications that make existing laws obsolete. Modifications, such as changes in head lights, tail lights, and vehicle widths have come about without apparent regard for existing statutes. Sales often rise to the tens of thousands before these conflicts are called to public attention. This situation has raised the question of how long must this sales directed disregard of vehicle codes be tolerated by public bodies.

A more costly area of concern is that of parking. Terminal facilities are becoming recongnized as an integral part of the highway problem. Brief mention will be made here of this subject.

The problem of the one car garage whose occupant outgrows the stable is well known. The problem is more costly with respect to commercial parking garages. The amount of parking space in such facilities depends largely on the column spacings and aisle widths of the original design. The increasing lengths, width, and turning radii of passenger vehicles have, in many cases, significantly reduced the capacity and revenue-earning ability of such facilities. The same is true of parking lots, although these can adjust more radily to the increased size of the passenger car. Even curb parking is affected by the increasing length of cars. The need to make parking spaces 22 ft long caused a 17% reduction in the number of spaces available, according to a recent survey in Pittsburgh.

A major question is - how much highway design is predicated on compensating for the lack of safety built into the vehicle and induced in the driver as a result of vehicle characteristics? It is felt that design considerations of median widths, median barriers, and guardrails are influenced by either or both of the above factors. In a recent report[4] these points are raised, (a) a better view of the right edge of pavement is needed, (b) more skid resistance between the pavement and the tire is needed. The report suggests that automobile design changes are the results of a continuing series of compromises between demands for styling and safety improvements. Safety improvements should improve the opportunity for the motorist to have control of the vehicle at all times and to react more effectively in times of an emergency.

Widening of pavements has been required to accomodate existing traffic more safely. There are times when it appears that the increased widths are used as justification by manufacturers and commercial groups to increase still further the size of their vehicles.

All of this adds up to a frustrating experience to the highway engineer. Highway design changes in the past have necessarily followed changes in vehicle design. But because the highway engineer is unaware of the manufacturers' longrange intentions, he has been building obsolesence into the highways at almost the same rate as the vehicle designer has been building obsolesence into the vehicle.

TODAY'S CRITICAL SITUATION

It is appropriate to review the present plight of highways and traffic operations resulting from the adjustments made to accommodate both vehicles and highways to the demands of society.

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[4] "The Federal Role in Highway Safety," House Document No. 93, Washington, D. C., 1959.

Whether the conditions existing on Delaware's highways are comparable to those of other states is difficult to say. But they are the conditions which have to be faced by at least one State Highway Department. Over the 512 miles of Delaware's primary highways, the system average of daily traffic exceeds 6500 vehicles. This traffic is carried on a network composed of 166 miles of 4 or 6 lane, divided highway, and at the other extreme, approximately 200 miles of two-lane highway, 22 ft or less in width. It is estimated from sufficiency studies that 247 miles need widening, exclusive of added lanes, a figure which includes mileage on divided highway lanes of 22 ft or less width. Despite the major emphasis in recent years on providing adequate pavement widths, it is obvious that construction has not yet caught up with the needs.

In terms of dollars, it is estimated that the cost of a two-lane primary rural route on new location is now $200,000 per mile. Taking into account deflation of the dollar, which is counter-balanced by improvements in construction productivity, this is a spectacular increase from the costs of $40,000 per mile for the same highway in the 1920's. Costs of major roadway items are listed in Table 4, which has been excerpted from a part of Delaware's response to the Section 210 Study. While some reductions in major cost items would result if design were based on 3000 lb axle loads, modern design, for passenger vehicles only, still results in costs far exceeding those of 30 years ago.

Truck weight studies in Delaware show that one-third of the commercial vehicles sampled in 1958 had gross weights of 45,000 lb or more. In 1941, there were no vehicles recorded with gross weights exceeding this figure. The average gross weight of all loaded vehicles in the 1958 survey was 46,000 lb, against a 1941 average of 32,000 lb. And the fact that applications for oversize or overweight vehicle movement permits have risen in Delaware approximately 600% in the past decade offers further evidence of the vehicle trends. In reply to a recent AASHO survey, 43 states noted an increasing number of permits for 10-ft wide house trailers, and 34 states remarked on the pressure existing to liberalize regulations. Of 14 states which permit movements of 12-ft wide trailers, 9 reported an increasing number of movements and pressure for an easing of restrictions.

The situation in current traffic operations poses some questions. The assortment of vehicles ranges in size from children's bicycles, through compact and "normal" cars, to 100 ft long, 65 ton freight vehicles. That there is at least segregation of the two extremes mentioned is something for which to be thankful. But the Delaware State Police recently investigated an accident on a high speed divided highway in which a tractor trailer had, without physical contact, turned over a small European car by the shock wave or back-wash while passing. The problems of being able to see the moving small car, the stopped school bus, or a changing traffic light when traveling behind, and preparing to overtake a tractor trailer are common experiences. The sight of columns of cars following a slow moving truck on an upgrade or even on flat terrain is not unusual. The possibility that exposed tandem wheels at the sides of trucks have an adverse psychological effect on small car drivers has been raised. None of these conditions are conducive to highway safety, and they give rise to public comment on the possibilities of separate roadways for 200,000 lb trucks and 1500 lb cars. It is a matter which merits study.

The cost of such investigations is small compared with the annual investments made by government units for highway purposes. Fig. 9 indicates the trend, in actual dollars spent, and in dollars adjusted to a 1957 base. In 1958 the total expenditures for all highway purposes, including maintenance and police, was approximately 9.8 billion dollars. The costs of future highway programs will be considerably higher if the growth of vehicle registrations contines at the rates shown in Fig. 10 for the past 10 years. The cost of the interstate system estimated in compliance with the 1956 Federal Highway Act, is over 32 billion dollars. The estimated cost to provide the Washington metropolitan area with a transportation system adequate for 1980 is 2.5 billion dollars. Another example of future costs can be seen in California's approved plan for the construction of 12,000 miles of freeways -- equivalent approximately 1/3 the National Interstate Highway System.

These programs will not be entered into lightly. Maximum knowledge of future needs is mandatory for judicious design. But can the designer feel assured, as he designs for the vehicle of today or even for those of the immediate future, that his creation will not be rendered obsolete by some new, unheralded departure in vehicle design? On the basis of past performance, he cannot.

TABLE 4.—VARIATIONS IN GEOMETRIC DESIGN STANDARDS AND COSTS FOR PRIMARY RURAL ROADS[a]

Items	Normal percentage of heavy vehicles		Light vehicles only Maximum axle load 3,000 pounds		% Difference	
(1)	2 Lane (2)	4 Lane (3)	2 Lane (4)	4 Lane (5)	2 Lane (6)	4 Lane (7)
Surface thickness	9 in.	9 in.	7 in.	7 in.	- 22	- 22
Surface width	24 ft	48 ft	22 ft	44 ft	- 8	- 9
Surface type	Concrete	Same	Same	Same	. . .	. . .
Shoulder width	10 ft	10 ft	8 ft	8 ft	- 20	- 20
Vertical clearance	14 ft	14 ft	10 ft	10 ft	- 29	- 29
Grading and drainage cost per mile	70,000	155,000	64,600	148,200	- 9	- 4
Shoulders cost per mile	1,700	6,900	1,300	5,500	- 24	- 20
Pavement and base cost per mile	89,000	178,000	69,500	139,000	- 22	- 22
Sub total	160,700	339,900	135,400	292,700	- 16	- 14
Interchange grading and drainage	105,000	120,000	. . .	92,000	. . .	- 23
Interchange shoulders	5,000	5,000	. . .	900	. . .	- 80

[a] Carrying light vehicles only and for roads carrying a normal percentage of heavy vehicles (flat terrain).

RESEARCH ACTIVITIES

The best measure of the concern of highway engineers regarding the problems of relating vehicle design to highway design is the amount of research activity which is currently underway. Within the Highway Research Board, the Department of Design, the Department of Economics, Finance and Administration, and

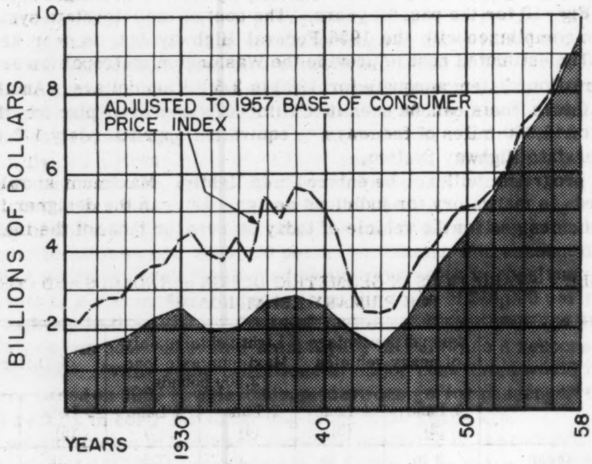

FIG. 9.—ANNUAL EXPENDITURES FOR HIGHWAY PORPOSES BY ALL UNITS OF GOVERNMENT SINCE 1922

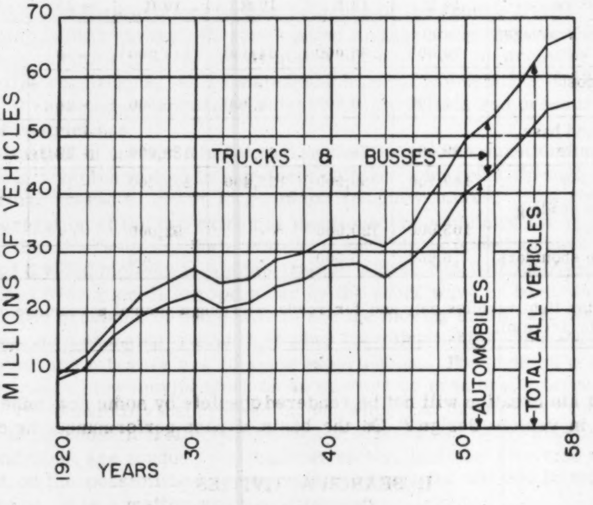

FIG. 10.—VEHICLE REGISTRATIONS 1920 TO 1958

the Department of Traffic and Operations are engaged, through various committee activities, in researching this problem. All told there are 20 committees of the Highway Research Board investigating areas in which the design of vehicles affects highway design. The American Society of Civil Engineers has a committee on geometrics of highway design and a committee on traffic engineering. In the American Society for Testing Materials a committee is being formed to investigate skid prevention. The American Association of State Highway Officials has at least five committees that are engaged in research related to this problem, the Operating Committees on Bridges and Structures, Design, Planning and Design Policies, Design, Construction, and Maintenance of Secondary Roads, and Traffic. The Technical Division of the Institute of Traffic Engineers has four departments whose activities must take into consideration the trends in vehicle design.

Generally, the activities of these committees represent the part-time efforts of highway and traffic engineers, most of whom are in public service. None of the committees are supplied with direct funds to engage in research. Research studies, when conducted, must be financed from Federal Highway Planning Survey funds and/or local budgets.

A listing of publicly or privately financed research projects has been obtained from the Highway Research Board. Twenty-seven subjects related directly or indirectly to problems of vehicle design and highway design can be tabulated. Many of these subjects are being studied by more than one agency. For example, under the heading of "Median Studies Related to Vehicle Accidents" studies are being conducted by the California Division of Highways, the New Jersey Turnpike Authority, and the New York Department of Public Works. Groups engaged in highway research include the automotive industry, various trade associations, the Bureau of Public Roads, State Highway Departments, and transportation research centers or other departments of many universities.

Other research investigations were instituted by the Federal Aid Highway Act of 1956. This act directed the Department of Commerce to undertake the so-called Section 210 Studies, which included investigation of (a) the effects on design, construction and maintenance of the use of vehicles of different dimensions, weights and specifications, (b) studies of the proportionate share of cost allocations, and (c) study of benefits derived from highway improvements. The Department of Commerce was also directed to undertake studies pertaining to improvements in highway safety. These studies have involved the efforts of the Bureau of Public Roads, the State Highway Departments, universities and other groups in developing the desired information over the past three years.

The major research activity, however, in terms of investment and scope has been the AASHO road test. In progress for the past 3 yr, this project is financed by the individual states, the Bureau of Public Roads, the Automobile Manufacturers Association Portland Cement Association and the American Petroleum Institute. The principal objective, reduced to its simplest form, is the determination of significant relationships between pavement performance and loads applied. Other objectives aim at providing related useful information to the highway designer and administrator. The total cost of this single study is estimated at approximately $22,000,000.00. Administration and direction for the project is being provided by the Highway Research Board of the National Academy of Sciences - National Research Council.

According to information supplied by the American Association of State Highway Officials, the amount spent in 1958 on all highway research activities, of which those described have been a part, was slightly under $18,000,000. The 1958 expenditure for all highway purposes by all units of government was slightly under $10,000,000,000. The research effort therefore is less than 2/10 of 1% of the total highway expenditures. Comparison of these figures with the research and development expenditures for private industry points up a noteworthy contrast. In the chemical industry, as a whole, it was estimated that in 1958 $600,000,000.00 or 3 to 4% of sales were spent on research. The 1958 annual report of E. I. DuPont de Nemours and Company, Inc., which is notably research minded, showed expenditures on research and development of $90,000,000 or 4.9% of gross sales.

The comparison is by no means justification for saying that highway research expenditures should be increased to the level of those of private industry. However, the significant difference which appears is, to say the least, disturbing. It is particularly disturbing when considering that the highway research effort outlined previously has been splintered among many diverse areas, and that the bulk of activities are committee operations involving the spare time activity of busy men in conferences, conventions, and reports for trade publications. It is fortunate that the efforts of these individuals have not been outstripped totally by the progress in vehicle improvements.

Is it enough to hope that the present nature of research activities can successfully meet the problems arising from vehicle design changes? Or will the following prophecy still apply?

> "There must be an arbitrary limit of load for which we can design our roads. Otherwise as fast as they are built, roads will attract to themselves traffic heavier than they are designed to bear. The road and the load will be forever outstripping each other with great economic loss both of original investment in the road and in the appalling high maintenance."

This warning was written in 1919 by H. E. Breed[5], then Deputy Commissioner of Highways for New York State. A more recent warning note was sounded by Robert Moses:[6]

> "The auto industry must play an increasingly important part. The period is over when the manufacturer loses interest when his car, truck or bus leaves the assembly line and the sales room-. . . There must be freer exchange of information, more discussion of difficulties, more inspections of successful installations and above all, better leadership."

CONCLUSIONS

The present reappraisal of the highway program being undertaken by Congress and other official bodies is occasioned by charges of poor coordination between engineers and planners, of unsound cost estimates, and of extravagant design. Yet one congressman recently stated that the prime example of overdesign on the American highway system is the automobile itself. The real question is: Can the highway engineers and the highways they design remain isolated from the many manufacturers and the private and public organizations which are dependent upon the highway systems for their very existence?

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[5] Public Roads, by H. E. Breed, January, 1919.

[6] Working for the People, by Robert Moses, Harper and Bros., New York, 1956.

The operation of a transportation system of the size and scope that exists today demands the best talent and the best thinking available. Even those who may feel that this is being provided must admit that research efforts are fragmented in terms of space, time, money, and manpower. We cannot afford to have our present and future investment in the National Interstate and Defense Highway System and other networks sacrificed through a lack of coordinated thought. Yet, common sense and engineering judgement dictate that a certain percentage of highway obsolescence must be accepted to achieve transportation progress. Common sense and engineering judgement, in the short term view, suggest also that the automotive industry and road user groups should defer major vehicle modifications until the findings of the 1956 Highway Act Studies and the AASHO Road Test can be made public.

An altogether too obvious solution to the problems which have been discussed is the establishment of further governmental bureaucracy to install controls, attempting thereby to protect the taxpayers' investment. Yet the very nature of such an agency would tend to hamper long range progress by restricting the competitive nature and creative thinking of a major industry and our own highway profession.

It is believed that an acceptable alternative is an expanded research program supported jointly by industry and all other interested agencies. This suggested new research effort will require a full time task force of workers in social and physical science disciplines as well as engineers from industry and public service. Equal emphasis must be given to the problems of manufacturers, users, and highway engineers.

In connection with the problem of correlating vehicle and highway design, certain functions should be assigned to such a research group:

1. Development of controlling maximum standards for vehicle design;
2. Determination of the maximum desirable life for any given set of standards;
3. Development of review procedures to permit periodic revision of standards;
4. Research to seek out new directions in highway transportation;
5. Determination of warrants for the installment of new system;
6. Dissemination of information to manufacturers, users, highway management, the public, and legislative bodies.

The precedent for such an organization has already been set. It is the AASHO Road Test, financed and directed by the major interested agencies and associations, public and private alike. The lessons in cooperation which have been learned as a result of this project can well be applied to the total problem at hand.

It is hoped that this paper will not be considered as merely self-indulgent tilting at the serenely rotating windmills of the automotive industry. This industry has a future no more sound than the foundations on which it is built. An integral part of the foundations is the national highway network. Like most foundations, highways are expensive, they must be conceived with a knowledge of the use to be made of them, and they call for optimum design to provide safe, efficient and economical service. Much more needs to be done by all concerned to achieve these ends.

APPENDIX.—ADDITIONAL REFERENCES

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5. "Handbook for Highway Engineers," by Wilson G. Harger, and Edmund A. Bonney, McGraw-Hill Book Co., Inc., New York, 1927.
6. "Driver Behavior as Related to Shoulder Type and Width on Two Lane Highways," by Asriel Taragin, Highway Research Bd. Bulletin 170, Washington, D. C. 1958.
7. "The Highway Cost Allocation Study," by G. P. St. Clair, Proceedings, ASCE, Vol. 85, No. HW1, January, 1959.
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ANALYSIS OF LOAD DISTRIBUTION IN THE PILES OF PIERS

By M. A. Gouda[1]

SYNOPSIS

Determination of the forces in piles of piers or a group of piles under the action of vertical, horizontal, and bending moment forces is outlined. By use of simplified assumptions and pile displacement, a method is developed for the analysis of load distribution on piles.

In a symmetrical pile group, forces are directly calculated, while in an unsymmetrical group the system is first treated as a symmetrical group and then by successive approximations (similar to moment-distribution methods) the forces are determined. A more exact solution for the problem is arrived at by considering the elastic deformation of piles together with the deck.

INTRODUCTION

Piers are mainly subjected to vertical forces due to loads from freight, cranes, vehicles, and the weight of the structure itself, as well as horizontal forces due to inward blows from ships berthing or to outward pull on mooring ropes. These forces, in some cases, will be the most serious factor affecting the design and the choice of a pile system. When the acting loads are not great, especially the horizontal force, vertical piles only are used.

On the other hand, when the horizontal force is of great intensity, the use of batter piles becomes necessary to provide sufficient lateral resistance and

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to increase the rigidity of the pier. Batter piles may be used alone or in combination with vertical piles.

PIERS WITH VERTICAL PILES ONLY

In the analysis of the load distribution on a group of vertical piles in a pier, the following assumptions are considered:

1. The deck of the pier (which may be considered as a pile cap) is rigid relative to the piles.
2. All piles are of the same cross section.
3. No settlement of the support of the piles occurs.
4. Points of support of the piles are at the same level.
5. Horizontal displacement of the top of each pile in relation to the point of embedment is the same for all piles.

In this type of pier the horizontal force is transferred to the ground by shear which produces bending moments on the pile. Hence the acting horizontal force must be small since the piles offer relatively little resistance to bending. For this reason, effective fenders as well as fender piles must be provided to absorb the shock and thus diminish the horizontal force causing the deflection.

Fig. 1 represents a pier on n vertical piles which is subjected to a vertical force V and horizontal force H. The loads on the piles are determined by a simple statical analysis.

If the group of piles in the pier is unsymmetrical and x is the distance of each pile from the centroid of the pile group and e is the eccentricity of the vertical force V from the axis of the centroid, the pier as a whole will be subjected to a moment V e - H h (e to right of centroid).

The axial load (P_x) on any pile is

$$P_x = \frac{V}{n} + \frac{x(V\,e - H\,h)}{\sum_1^h x^2} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

According to assumption 5, the horizontal force H is resisted equally by each pile and the shearing force on any pile will be

$$Q_x = \frac{H}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Assuming fixity at the top or datum level of pile, the bending moment will be

$$M_x = \frac{H\,h}{2\,n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Example 1.—Given a symmetrical pier with three vertical piles subjected to vertical and horizontal forces (Fig. 2):

Apply Eqs. 1, 2, and 3 to the pier:

Since V is symmetrical, e = 0 and

$$P_1 = \frac{150}{3} - 7.5\left(\frac{100}{2\ (7.5)^2}\right)$$

$$= -\ 50\ -\ 6.67 = -\ 56.67 \text{ tons}$$

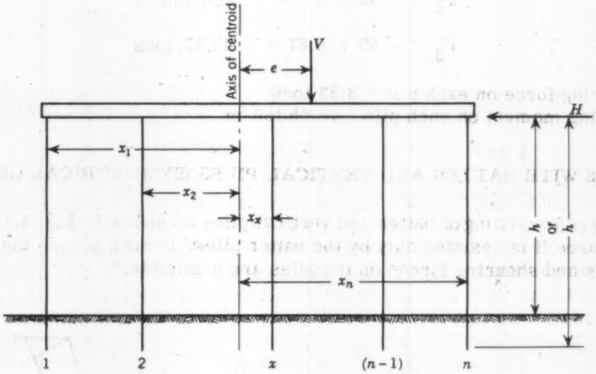

Axis of centroid
V
e
H
x_1
x_2
x_x
x_n
h or h
1
2
x
(n−1)
n

FIG. 1

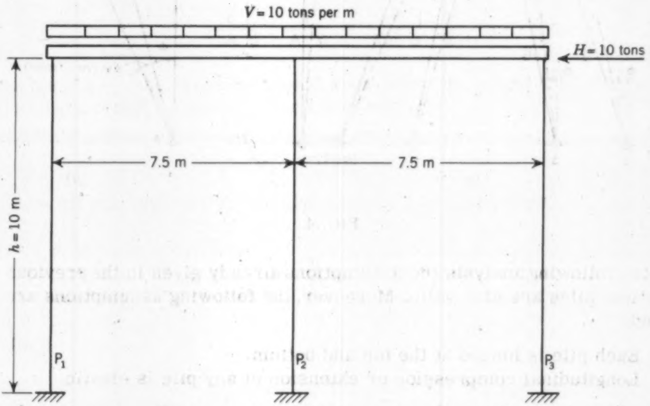
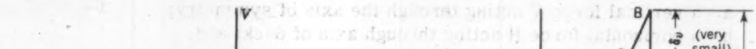

V = 10 tons per m
H = 10 tons
7.5 m
7.5 m
h = 10 m
P1
P2
P3

FIG. 2


V
δ_v
v_1
v_2
v_n
P1
P2
ϕ_2
ϕ_2
h
1
2
3
x
(n−1)
n
B
C
B′
δ_v (very small)
ϕ
$\delta\phi$ (very small)
A
h

(a) (b)

FIG. 3

$$P_2 = -\ 50\ -\ 0 \quad = -\ 50.0 \text{ tons}$$

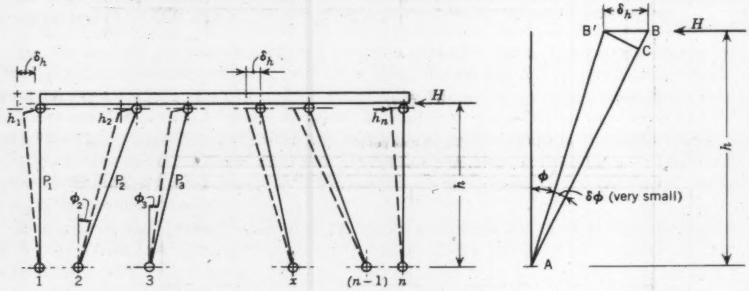
and

$$P_3 = -\ 50 + 6.67 = -\ 43.33 \text{ tons}$$

Shearing force on each pile = 3.33 tons
Bending moment on each pile = 16.65 ton-m

PIERS WITH BATTER AND VERTICAL PILES (SYMMETRICAL GROUP)

In piers consisting of batter and vertical piles as shown in Fig. 4, the horizontal force H is resisted only by the batter piles. In such groups the bending moments and shearing forces on the piles are negligible.



(a) (b)

FIG. 4

In the following analysis the assumptions already given in the previous case of vertical piles are still valid. Moreover, the following assumptions are considered:

1. Each pile is hinged at the top and bottom.
2. Longitudinal compression or extension of any pile is elastic.

Fig. 3(a) shows a pier of a symmetrical group of n piles inclined at ϕ_1, ϕ_2, etc., each having the same dimensions, elastic constant, and height h.

In piers, any system of forces acting could be reduced to

a. a vertical force V acting through the axis of symmetry;
b. a horizontal force H acting through axis of deck; and
c. a moment M.

A Pier Subjected to a Vertical Force V.—If a vertical force acts on a symmetrical pile pier at its centroidal axis, as shown in Fig. 3(a), and since the deck is rigid relative to the piles, the vertical displacement δ_v at the top of each pile will be the same. Thus

$$\delta_{v_1} = \delta_{v_2} = \ldots \delta_{v_n} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

The vertical force V will be resisted by vertical components at the top of the pile such that

$$V = v_1 + v_2 + \ldots\ldots\ldots + v_n \ldots\ldots\ldots\ldots\ldots \quad (5)$$

If the axial force in each pile is P_1, P_2, P_3, etc., then

$$V = P_1 \cos \phi_1 + P_2 \cos \phi_2 + \ldots\ldots + P_n \cos \phi_n \ldots\ldots \quad (6)$$

Consider a pile AB inclined at an angle ϕ as shown in Fig. 3(b); when subjected to a vertical force V, the pile moves to AB'.

The vertical displacement (δ_v) can be regarded as a displacement $\delta_v \sin \phi$ (represented by CB') at right angles to AB and a contraction $\delta_v \cos \phi$ (represented by BC). Since the displacements due to load are very small compared with the length of the pile then

$$AB \triangleq AC = \frac{h}{\cos \phi} \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

The total contraction of the pile is given by

$$\frac{P}{\cos \phi} \; \frac{h}{A\,E} = \frac{P}{\cos \phi} \; \frac{h}{k} \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

in which k = AE = the cross-sectional area times the modulus of elasticity.

The vertical displacement at the top of any pile

$$\delta_v = \frac{P}{\cos^2 \phi} \; \frac{h}{k} \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

and since all pile tops are vertically displaced the same then

$$\frac{P_1}{\cos^2 \phi_1} \; \frac{h}{k} = \frac{P_2}{\cos^2 \phi_2} \; \frac{h}{k} = \ldots\ldots = \frac{P_n}{\cos^2 \phi_n} \; \frac{h}{k} \ldots\ldots \quad (10)$$

therefore

$$P_2 = P_1 \; \frac{\cos^2 \phi_2}{\cos^2 \phi_1} = \ldots\ldots = P_1 \; \frac{\cos^2 \phi_n}{\cos^2 \phi_1} \ldots\ldots \quad (11)$$

From Eq. 6

$$V = P_1 \left(\cos \phi_1 + \frac{\cos^3 \phi_2}{\cos^2 \phi_1} + \ldots\ldots + \frac{\cos^3 \phi_n}{\cos^2 \phi_1} \right) \ldots\ldots \quad (12)$$

hence

$$P_1 = V \; \frac{\cos^2 \phi_1}{\cos^3 \phi_1 + \cos^3 \phi_2 + \ldots\ldots + \cos^3 \phi_n} \ldots\ldots \quad (13)$$

and generally the axial force in any pile due to the vertical force V is

$$P_x = V \frac{\cos^2 \phi_x}{\sum_1^n \cos^3 \phi_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

A Pier Subjected to a Horizontal Force H.—If a horizontal force H acts at the deck of the pier as shown in Fig. 4(a) and since the pile deck is rigid

$$\delta_{h_1} = \delta_{h_2} = \ldots\ldots = \delta_{h_n} \quad \ldots\ldots\ldots\ldots (15)$$

and the horizontal force H is opposed by horizontal reactions h_1, h_2, h_3, etc., from the top of each pile

$$H = h_1 + h_2 + \ldots\ldots + h_n$$

therefore

$$H = P_1 \sin \phi_1 + P_2 \sin \phi_2 + \ldots\ldots + P_n \sin \phi_n \quad \ldots\ldots (16)$$

In Fig. 4(b) the pile AB, when subjected to a horizontal force H, moves to AB' which could be considered as a displacement $\delta_h \cos \phi$ at right angles to AB and a contraction (or extension if pile is inclined toward the left) $\delta_h \sin \phi$.

The horizontal displacement at the top of any pile

$$\delta_h = \frac{P}{\cos \phi \sin \phi} \frac{h}{k} \quad \ldots\ldots\ldots\ldots\ldots (17)$$

and from Eq. 15

$$\frac{P_1}{\cos \phi_1 \sin \phi_1} \frac{h}{k} = \frac{P_2}{\cos \phi_2 \sin \phi_2} \frac{h}{k} = \ldots = \frac{P_n}{\cos \phi_n \sin \phi_n} \frac{h}{k} \quad \ldots (18)$$

and by expressing axial pile forces P_2, P_3, etc. in terms of P_1

$$P_2 = P_1 \frac{\cos \phi_2 \sin \phi_2}{\cos \phi_1 \sin \phi_1} \quad \ldots\ldots\ldots\ldots\ldots (19a)$$

and

$$P_n = P_1 \frac{\cos \phi_n \sin \phi_n}{\cos \phi_1 \sin \phi_1} \quad \ldots\ldots\ldots\ldots\ldots (19b)$$

By substituting for P_2, P_3 in Eq. 16 the horizontal force

$$H = P_1 \sin \phi_1 + \frac{P_1 \cos \phi_2 \sin^2 \phi_2}{\cos \phi_1 \sin \phi_1} + \ldots\ldots + \frac{P_1 \cos \phi_n \sin^2 \phi_n}{\cos \phi_1 \sin \phi_1} \ldots (20)$$

therefore

$$H = P_1\left(\frac{\cos\phi_1\sin^2\phi_1 + \cos\phi_2\sin^2\phi_2 + \ldots + \cos\phi_n\sin^2\phi_n}{\cos\phi_1\sin\phi_1}\right) \quad . . (21)$$

Hence

$$P_1 = H\left(\frac{\cos\phi_1\sin\phi_1}{\cos\phi_1\sin^2\phi_1 + \cos\phi_2\sin^2\phi_2 + \ldots + \cos\phi_n\sin^2\phi_n}\right) \quad . . (22)$$

In general the axial force in any pile due to a horizontal force H

$$P_x = H\,\frac{\cos\phi_x\sin\phi_x}{\sum_1^n \cos\phi_x\sin^2\phi_x} \quad \ldots\ldots\ldots\ldots (23)$$

Axial Forces in Piles Due to a Bending Moment M.—If the deck of a symmetrical pile pier, as shown in Fig. 5(a) is subjected to a bending moment M it will rotate through an angle α. This bending moment may be due to the eccentricity of the acting vertical force V or the horizontal force H. In this case a counter-clockwise moment will develop a compression on piles of the left-hand group and tension on the right-hand group and vice versa for a clockwise moment. It must be noticed that the sums of the resulting vertical components of the forces on the piles and of the corresponding horizontal components are both zero.

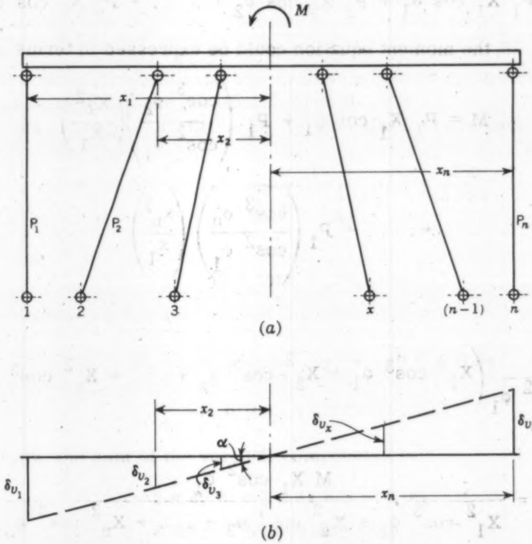


FIG. 5

Since the pier deck is rigid in comparison to the piles, the vertical displacement of each pile will be in proportion to its distance from the axis of symmetry.

Referring to Fig. 5(b),

$$\frac{\delta_{v_1}}{\delta_{v_2}} = \frac{X_1}{X_2} \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots \text{(24)}$$

By using the expression of the vertical displacement from Eq. 9

$$\frac{P_1}{P_2} \; \frac{\cos^2 \phi_2}{\cos^2 \phi_1} = \frac{X_1}{X_2} \quad \dots\dots\dots\dots\dots\dots\dots \text{(25)}$$

therefore

$$P_2 = P_1 \; \frac{\cos^2 \phi_2}{\cos^2 \phi_1} \; \frac{X_2}{X_1} \quad \dots\dots\dots\dots\dots\dots\dots \text{(26)}$$

The external moment M will be resisted by the vertical components of the reactions at the top of the piles in such a way that

$$M = P_1 X_1 \cos \phi_1 + P_2 X_2 \cos \phi_2 + \dots + P_n X_n \cos \phi_n \dots \text{(27)}$$

From Eq. 26, the moment equation could be expressed in terms of P_1 only:

$$M = P_1 X_1 \cos \phi_1 + P_1 \left(\frac{\cos^3 \phi_2}{\cos^2 \phi_1}\right)\left(\frac{X_2{}^2}{X_1}\right)$$

$$+ \dots + P_1 \left(\frac{\cos^3 \phi_n}{\cos^2 \phi_1}\right)\left(\frac{x_n{}^2}{x_1}\right) \quad \dots\dots\dots\dots \text{(28)}$$

therefore

$$M = \frac{P_1}{X_1 \cos^2 \phi_1}\left(X_1{}^2 \cos^3 \phi_1 + X_2{}^2 \cos^3 \phi_2 + \dots + X_n{}^2 \cos^3 \phi_n\right) \dots \text{(29)}$$

and

$$P_1 = \frac{M X_1 \cos^2 \phi_1}{X_1{}^2 \cos^3 \phi_1 + X_2{}^2 \cos^3 \phi_2 + \dots + X_n{}^2 \cos^3 \phi_n} \quad \dots \text{(30)}$$

and, in general, the axial force in any pile due to an external moment M is given by

$$P_x = M \; \frac{X_x \cos^2 \phi_x}{\sum_1^n X_x^2 \cos^3 \phi_x} \quad \dots\dots\dots\dots \text{(31)}$$

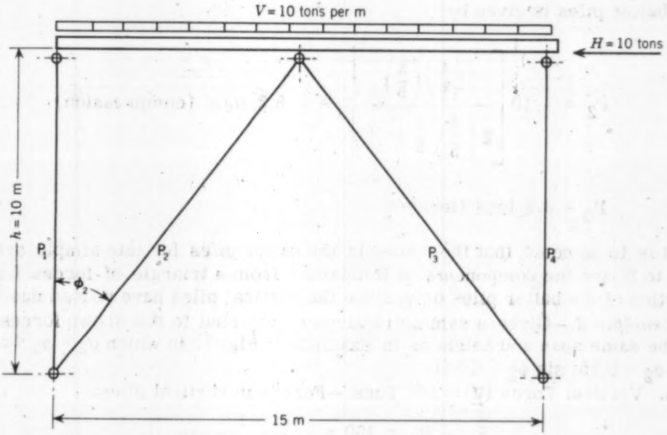


FIG. 6

Example 2.—Given a simple symmetrical pier of a group of batter and vertical piles (Fig. 6) in which

$$\phi_1 = \phi_4 = 0; \ \phi_2 = \phi_3 \quad (\text{where } \cos\phi_2 = 4/5; \ \sin\phi_2 = 3/5)$$

In the following, the pile axial forces are calculated separately for each of the vertical force V and the horizontal force H.

a. Vertical Forces (V).—From Eq. 14 the axial force in each of the vertical piles:

$$P_1 = P_4 = -150 \ \frac{1}{2\left[1 + \left(\frac{4}{5}\right)^3\right]} = -\frac{150}{3.024}$$

$$= -\ 49.6 \text{ tons (compression)}$$

The corresponding force in each of the batter piles:

$$P_2 = P_3 = -49.6 \ \left(\cos^2 \phi_2\right) = -49.6 \times 0.64$$

$$= -31.8 \text{ tons (compression)}$$

As a check, the sum of the vertical components is:

$$2 \times 49.6 + 2 \times 31.8 \times \cos\phi_2$$
$$= 99.2 + 50.8 = 150 \text{ tons}$$

b. Horizontal Force H = 10 Tons.—There is no force in the vertical piles (P_1, P_4) due to the horizontal force H. From Eq. 23 the axial force in each of the batter piles is given by

$$P_2 = -10\left[\frac{\left(\frac{4}{5}\right)\left(\frac{3}{5}\right)}{2\left[\frac{4}{5}\left(\frac{3}{5}\right)^2\right]}\right] = -8.3 \text{ tons (compression)}$$

$$P_3 = 8.3 \text{ tons (tension).}$$

It is to be noted that the forces in the batter piles for this simple system due to H are the components of H obtained from a triangle of forces for the junction of the batter piles only, since the vertical piles have no load due to H:

Example 3.—Given a symmetrical pier subjected to the shown forces and of the same span and height as in Example 2 (Fig. 7) in which $\phi_2 = \phi_3$ (where $\cos\phi_2 = 0.95$; $\sin\phi_2 = 0.31$)

a. Vertical Force (V) = 150 Tons.—Forces in vertical piles:

$$P_1 = P_4 = 150 \times \frac{1}{2 + 2 \times 0.857}$$

$$= -40.5 \text{ tons}$$

Forces in batter piles:

$$P_2 = P_3 = 40.5 \times \cos^2\phi_2$$

$$= 40.5 \times 0.9$$

$$= -36.5 \text{ tons}$$

b. Horizontal Force (H) = 10 Tons.—Forces in vertical piles:

$$P_1 = P_4 = 0.$$

Forces in batter piles:

$$P_2 = -10 \times \frac{0.95 \times 0.31}{2\left[0.95 \times (0.31)^2\right]}$$

$$= -16.1 \text{ tons (compression).}$$

$$P_3 = +16.1 \text{ tons (tension).}$$

Example 4.—Fig. 8 shows a pier of a symmetrical group consisting of two vertical piles and four batter ones:

$$\phi_2 = \phi_5 \text{ (where } \cos\phi_2 = 0.943;\ \sin\phi_2 = 0.33)$$

$$\phi_3 = \phi_4 \text{ (where } \cos\phi_3 = 0.957;\ \sin\phi_3 = 0.286)$$

a. Vertical Forces (V).—Forces in vertical piles:

$$P_1 = P_6 = 150 \times \frac{1}{2 + 2 \times 0.827 + 2 \times 0.88}$$

$$= \frac{150}{5.414} = -27.8 \text{ tons}$$

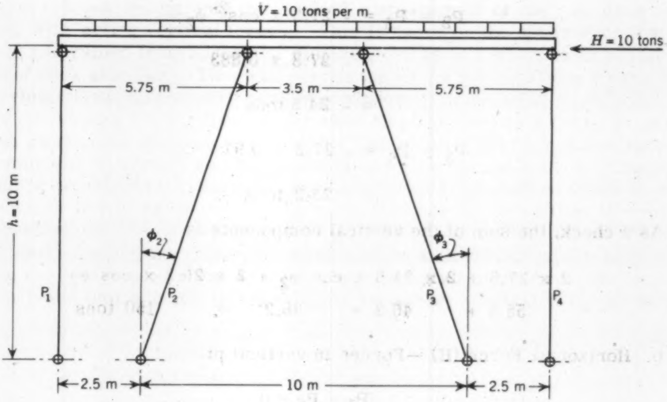

V = 10 tons per m
H = 10 tons
5.75 m
3.5 m
5.75 m
h = 10 m
ϕ_2
ϕ_3
P_1
P_2
P_3
P_4
2.5 m
10 m
2.5 m

FIG. 7

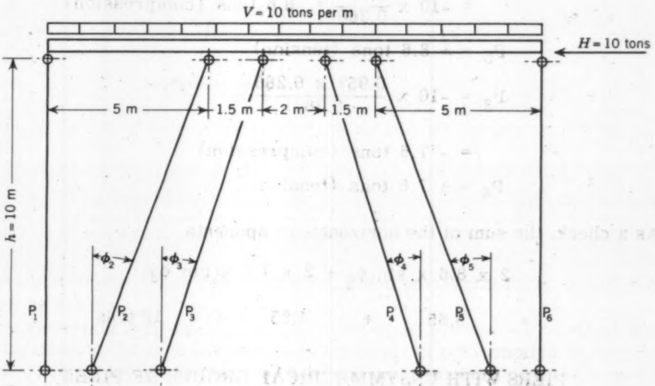

V = 10 tons per m
H = 10 tons
5 m
1.5 m
2 m
1.5 m
5 m
h = 10 m
ϕ_2
ϕ_3
ϕ_4
ϕ_5
P_1
P_2
P_3
P_4
P_5
P_6
2.5 m
1.5 m
2.5 m
1.5 m

FIG. 8

Forces in batter piles:

$$P_2 = P_5 = -\ 27.8 \text{ x } \cos^2 \phi_2$$

$$= -\ 27.8 \text{ x } 0.883$$

$$= -\ 24.5 \text{ tons}$$

$$P_3 = P_4 = -\ 27.8 \text{ x } 0.91$$

$$= -\ 25.3 \text{ tons}$$

As a check, the sum of the vertical components is

$$2 \text{ x } 27.8 + 2 \text{ x } 24.5 \text{ x } \cos \phi_2 + 2 \text{ x } 25.3 \text{ x } \cos \phi_3$$

$$= 55.6 + 46.2 + 48.2 = 150 \text{ tons}$$

b. Horizontal Force (H).—Forces in vertical piles:

$$P_1 = P_6 = 0$$

Forces in batter piles:

$$P_2 = -10 \text{ x } \frac{0.943 \text{ x } 0.33}{2\text{x}0.943 \text{ x } 0.108 + 2\text{x}0.957 \text{ x } 0.082}$$

$$= -10 \text{ x } \frac{0.312}{0.360} = -8.6 \text{ tons (compression)}$$

$$P_5 = +\ 8.6 \text{ tons (tension)}$$

$$P_3 = -10 \text{ x } \frac{0.957 \text{ x } 0.286}{0.36}$$

$$= -\ 7.6 \text{ tons (compression)}$$

$$P_4 = +\ 7.6 \text{ tons (tension)}.$$

As a check, the sum of the horizontal components

$$2 \text{ x } 8.6 \text{ x } \sin \phi_2 + 2 \text{ x } 7.6 \text{ x } \sin \phi_3$$

$$= 5.65 + 4.35 = 10 \text{ tons}$$

PIERS WITH UNSYMMETRICAL GROUPS OF PILES

In the analysis of axial forces on piles in an unsymmetrical group it is necessary to find the elastic center of the piles. This is defined as the point through which a vertical or horizontal force is acting which will cause only translation displacement without rotation. To find this point and the final forces in piles, several equations must be derived and the group of piles must be divided into left hand and right hand groups.

In the following there is determined a simple and practical solution for the problem by considering only the vertical displacement of the pier deck when dealing with acting vertical forces. This is based on the assumption that the deck is restrained from horizontal displacement as in a wharf or by neglecting effect of deck side sway. This will permit the use of the equations for the case of a symmetrical pier as a first approximation.

By introducing an imaginary horizontal force at the pier's deck, the effect of side sway could be taken into account. The value of this horizontal force is determined from laws of equilibrium applied to the pier as a whole. The equilibrium of horizontal reactions will create a residual unbalanced small vertical force which has to be distributed on piles, similar to the method of moment distribution.

A similar procedure is adopted when dealing with horizontal forces by neglecting vertical displacement as a first approximation in the first instance and is then taken into account by introducing imaginary vertical force to satisfy laws of equilibrium. The resulting unbalanced horizontal force is distributed on piles as before.

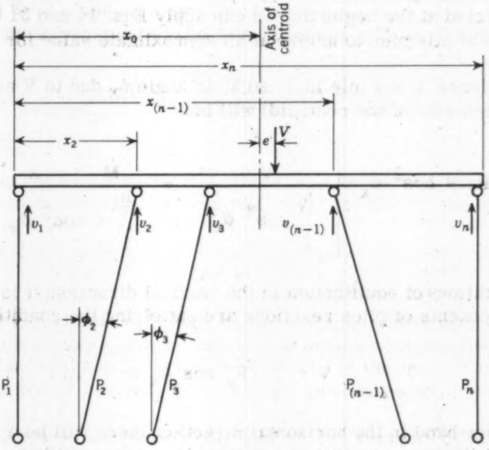


FIG. 9

Unsymmetrical Pile Pier Subjected to Vertical Forces.—If vertical forces are to be considered, it is assumed that the effect of horizontal movement (deck side sway) is neglected in the first instance and that each pile is compressed vertically by the same amount. This is the case when the resultant of the acting vertical forces V is passing through the axis of the piles' centroid. Otherwise the pile deck will rotate beside vertical movement due to moment M = V e which has to be considered in the calculation (Fig. 9).

Determination of Axis of the Centroid.—It is assumed that the pile deck in Fig. 9 is displaced a unit distance vertically downwards. Referring to Fig. 3(b) the axial deformation of pile AB is $\cos \phi$ and the reaction of the pile is $\cos^2 \phi \, \frac{k}{h}$ (where $k = A\ E$).

The vertical component of the reaction to the top of pile AB is equal to (k/h) $(\cos^3 \phi)$. Denoting the distance of the centroid of the vertical components of piles reactions from the left-hand pile by X_0

$$X_0 = \frac{x_1 \cos^3 \phi_1 + x_2 \cos^3 \phi_2 + \ldots + x_n \cos^3 \phi_n}{\cos^3 \phi_1 + \cos^3 \phi_2 + \ldots + \cos^3 \phi_n} \quad \ldots \ldots (32)$$

from which

$$X_0 = \frac{\sum_1^n x \cos^3 \phi_x}{\sum_1^n \cos^3 \phi_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Forces in Piles by Successive Approximation.—Since the effect of deck side sway is neglected at the beginning, we can apply Eqs. 14 and 31 for the case of the symmetrical pile pier to arrive at an approximate value for the pile reactions.

The axial force in any pile inclined at an angle ϕ_x due to V and M (V is not passing through axis of the centroid) will be:

$$P_x = \cos^2 \phi_x \left[\frac{V}{\sum_1^n \cos^3 \phi_x} + \frac{M \quad x}{\sum_1^n x \cos^3 \phi_x} \right] \quad \ldots\ldots (34)$$

From conditions of equilibrium in the vertical direction, it is found that the vertical components of piles reactions are satisfying the condition

$$V = \sum_1^n P_x \cos \phi_x \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

On the other hand in the horizontal direction, there will be a residual horizontal force. This means that the system is not in equilibrium in the latter direction and an imaginary horizontal force H must be applied at the pier deck where

$$H = - \sum P_x \sin \phi_x \quad \ldots\ldots\ldots\ldots\ldots (36)$$

This horizontal force will create additional axial forces in the piles.

By considering the effect only of horizontal movement (vertical displacement is neglected) in calculating pile reactions due to the imaginary deck force H, one could apply Eq. 23 as in the case of a symmetrical pile pier. The additional axial force on any pile inclined at an angle ϕ_x due to H will be

$$P_x = H \frac{\cos \phi_x \sin \phi_x}{\sum_1^n \cos \phi_x \sin^2 \phi_x} \quad \ldots\ldots\ldots (37)$$

A close examination of the algebraic sum of the horizontal components of pile reactions (obtained by adding results from Eqs. 34 and 37 shows that they are equal to the imaginary deck force H while vertical components will not equal the acting force V. This means that we have to repeat the cycle back again with a new imaginary vertical force

$$V = - \sum_{1}^{n} P_x \cos \phi_x \quad \ldots\ldots\ldots\ldots\ldots \quad (38)$$

Actually this last imaginary force V is small and will create a very small unbalanced horizontal force H, so that it could be neglected. Otherwise the process is repeated until the required precision is attained.

Combining results from all previous acting and imaginary deck forces, the final axial pile reactions are determined for the case of the pier when it is free to sway.

Unsymmetrical Pile Pier Subjected to Horizontal Forces.—The procedure for this case of acting horizontal forces is the same as the previous one. First, the vertical displacement is neglected to get approximate values of pile reactions by treating the system as a symmetrical one.

From Eq. 23 the approximate axial pile reactions will be:

$$P_x = H \frac{\cos \phi_x \sin \phi_x}{\sum_{1}^{n} \cos \phi_x \sin^2 \phi_x} \quad \ldots\ldots\ldots\ldots\ldots \quad (39)$$

Resolving pile reactions vertically, there will be an unbalanced vertical force which has to be distributed over piles by considering the effect of an imaginary vertical force

$$V = - \sum_{1}^{n} P_x \cos \phi_x \quad \ldots\ldots\ldots\ldots\ldots \quad (40)$$

This imaginary vertical force will create back an unbalanced horizontal force determined from laws of equilibrium. It must be noticed that the residual unbalanced forces are progressively decreased as the process is repeated. Algebraic summation of results from previous processes will give the final pile reactions.

Example 5.—Given an unsymmetrical pier of two vertical piles and three batter ones (Fig. 10) in which

$$\phi_2 = \phi_4 \quad (\text{where } \cos \phi_2 = 0.943; \ \sin \phi_2 = 0.33;$$

$$\cos \phi_3 = 0.957; \ \sin \phi_3 = 0.286)$$

The loadings, span and height of pier are similar to previous example.

Axis of the Centroid (Fig. 11).—From Eq. 33

$$x_0 = \frac{5 \cos^3 \phi_2 + 6.5 \cos^3 \phi_3 + 10 \cos^3 \varphi_4 + 15}{1 + \cos^3 \phi_2 + \cos^3 \phi_3 + \cos^3 \phi_4 + 1}$$

$$= 7.3 \text{ m}$$

$$\text{Eccentricity, } e = 0.20 \text{ m}$$

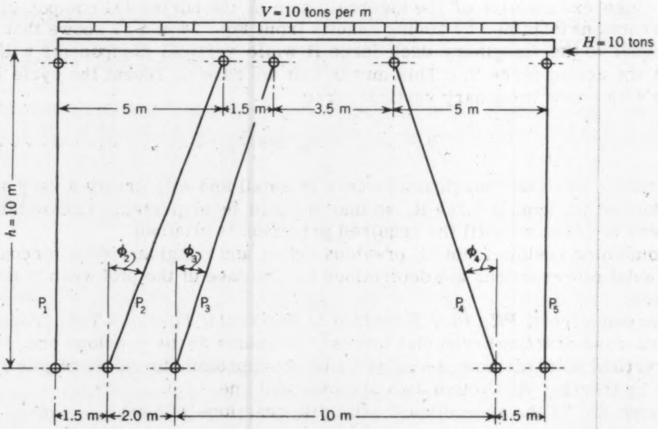

V = 10 tons per m
H = 10 tons
5 m
1.5 m
3.5 m
5 m
h = 10 m
ϕ_2
ϕ_3
ϕ_4
P_1
P_2
P_3
P_4
P_5
1.5 m
2.0 m
10 m
1.5 m

FIG. 10

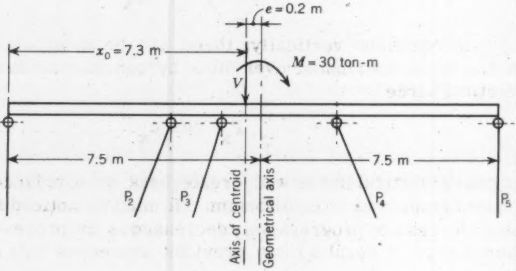

e = 0.2 m
x_0 = 7.3 m
M = 30 ton-m
V
7.5 m
7.5 m
P_1
P_2
P_3
P_4
P_5
Axis of centroid
Geometrical axis

FIG. 11

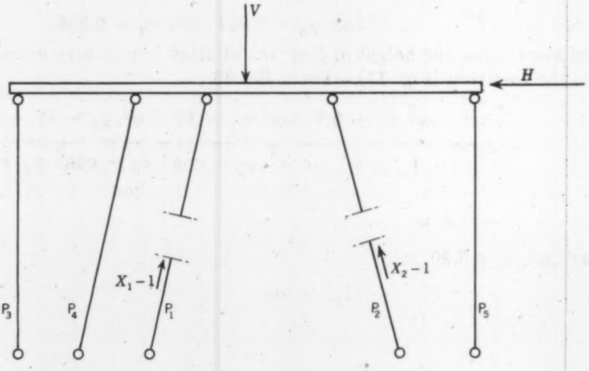

V
H
$X_1 - 1$
$X_2 - 1$
P_3
P_4
P_1
P_2
P_5

FIG. 12

Thus the pier deck will be subjected to a vertical force (V = 150 tons) and moment (M = 150 x 0.2 = 30 ton-m) passing through axis of centroid as shown in Fig. 11.

a. Forces in Piles Due to V or H or M.—Eqs. 14, 23, and 31 are used to determine the effect of V, H, or M. A solution is presented in Tables 1 and 2.

MORE EXACT SOLUTION

A more exact solution for symmetrical and unsymmetrical groups of piles may be approached by considering the group together with their deck, as a statistically indeterminate system. A main statically determinate system is to be first assumed and then the unknown axial forces in the redundant piles can be calculated by the method of virtual work. This method has the advantage of introducing the bending and axial deformations in the pile deck, as well as the axial deformations of the piles to the solution.

Moreover, this method can be applied to the same degree of accuracy for symmetrical and unsymmetrical groups of piles. As the deck of the pier is always compared to the piles, the assumption of hinged connections between the deck and the piles will help to decrease the statical indeterminacy of the system.

In Fig. 12 the main system is chosen by assuming a cut in piles 1 and 2 and calculating the bending moment M_0 and thrust N_0 diagrams from the equations of statical equilibrium. Virtual loads $X_1 = 1$ and $X_2 = 1$ are then applied along the assumed cuts. Diagrams for M_1, N_1, M_2 and N_2 are then drawn. The redundant axial forces in piles 1 and 2 can be determined from the following equations of deformation at the assumed cuts:

$$\delta_{10} + X_1 \delta_{11} + X_2 \delta_{12} = \frac{X_1 L_1}{A_1 E} \quad \ldots\ldots\ldots\ldots (41)$$

and

$$\delta_{20} + X_1 \delta_{21} + X_2 \delta_{22} = \frac{X_2 L_2}{A_2 E} \quad \ldots\ldots\ldots\ldots (42)$$

where L_1 and L_2 are the lengths of piles P_1 and P_2 and A_1 and A_2 are the areas of piles P_1 and P_2.

The deformations are given by

$$\delta_{10} = \int \frac{M_0 M_1}{I E} dl + \sum \frac{N_0 N_1}{A E} \Delta l \quad \ldots\ldots\ldots (43)$$

and

$$\delta_{11} = \int \frac{M_1^2}{I E} dl + \sum \frac{N_1^2}{A E} \Delta l \quad \ldots\ldots\ldots\ldots (44)$$

TABLE 1.—FORCE DISTRIBUTION DUE TO VERTICAL LOAD IN FIG. 10

Loading (1)	Pile (2)	$P_{V'}$ in tons (3)	$P_{H'}$ in tons (4)	$P_{M'}$ in tons (5)	P_{TOTAL}, in tons (6)	Vertical Component, Col. 6 cos ϕ (7)	Horizontal Component, Col. 6 sin ϕ (8)
(a) Analysis							
Case I:							
Vertical =	1	-33.1	. . .	1.78	-31.32	31.32 ↑	0
150 tons ↓	2	-29.2	. . .	0.495	-28.705	27.1 ↑	9.45 →
	3	-30.1	. . .	0.177	-29.923	28.6 ↑	8.55 →
Moment =	4	-29.2	. . .	-0.582	-29.782	28.1 ↑	9.8 ←
30 ton-m ↷	5	-33.1	. . .	-1.88	-34.98	34.98 ↑	0
					Total	150.1 ↑	8.2 → Unbalanced
Case II:							
Horizontal =	1	. . .	. . .	. . .	. . .	0	0
8.2 tons →	2	. . .	9	. . .	9	8.5 ↑	2.96 ←
	3	. . .	8	. . .	8	7.62 ↓	2.28 ←
	4	. . .	-9	. . .	-9	8.5 ↑	2.96 ←
	5	. . .	. . .	. . .	. . .	0	0
					Total	7.62 ↓ Unbalanced	8.2 ←
Case III:							
Vertical =	1	-1.69	. . .	. . .	-1.69	1.69 ↑	0
7.62 tons ↓	2	-1.485	. . .	. . .	-1.485	1.4 ↑	0.49 →
	3	-1.5	. . .	. . .	-1.5	1.4 ↑	0.43 →
	4	-1.485	. . .	. . .	-1.485	1.4 ↑	0.49 ←
	5	-1.69	. . .	. . .	-1.69	1.69 ↑	0
					Total	7.62 ↑	0.43 → Unbalanced

(b) Final Pile Forces

Pile	Case I	Case II	Case III	Pile Force, in tons
1	-31.32	0	-1.69	-33.01
2	-28.705	9	-1.485	-21.19
3	-29.923	8	-1.5	-23.423
4	-29.782	-9	-1.485	-40.267
5	-34.98	0	-1.69	-36.67

TABLE 2.—FORCE DISTRIBUTION DUE TO HORIZONTAL LOAD IN FIG. 10

Loading (1)	Pile (2)	P_V, in tons (3)	P_H, in tons (4)	P_M, in tons (5)	P_{TOTAL}, in tons (6)	Vertical Component, Col. 6 cos ϕ (7)	Horizontal Component, Col. 6 sin ϕ (8)
				(a) Analysis			
Case I:	1	. . .	. . .	. . .	. . .	0	0
Horizontal =	2	. . .	-11.0	. . .	-11.0	10.4 ↑	3.625→
10 tons ←	3	. . .	- 9.7	. . .	- 9.7	9.3 ↑	2.75 →
	4	. . .	11.0	. . .	11.0	10.4 ↓	3.625→
	5	. . .	. . .	. . .	. . .		
					Total	9.3 ↑ Unbalanced	10.0 →
Case II:	1	2.06	. . .	. . .	2.06	2.06 ↓	0
Vertical =	2	1.81	. . .	. . .	1.81	1.71 ↓	0.6 ←
9.3 tons ↑	3	1.83	. . .	. . .	1.83	1.76 ↓	0.525←
	4	1.81	. . .	. . .	1.81	1.71 ↓	0.6 →
	5	2.06	. . .	. . .	2.06	2.06 ↓	
					Total	9.3 ↓	0.525← Unbalanced
Case III:	1	. . .	. . .	. . .	. . .	0	0
Horizontal =	2	. . .	-0.58	. . .	-0.58	0.548 ↑	0.191→
0.525 ton ←	3	. . .	-0.51	. . .	-0.51	0.486 ↑	0.145→
	4	. . .	0.58	. . .	0.58	0.548 ↓	0.191→
	5	. . .	. . .	. . .	. . .		0
					Total	0.486 ↑ Unbalanced	0.527→

(b) Final Pile Forces

Pile	Case I	Case II	Case III	Pile Force, in tons
1	0	2.06	0	2.06
2	-11.0	1.81	-0.58	-9.77
3	- 9.7	1.83	-0.51	-8.38
4	11.0	1.81	0.58	13.39
5	0	2.06	0	2.06

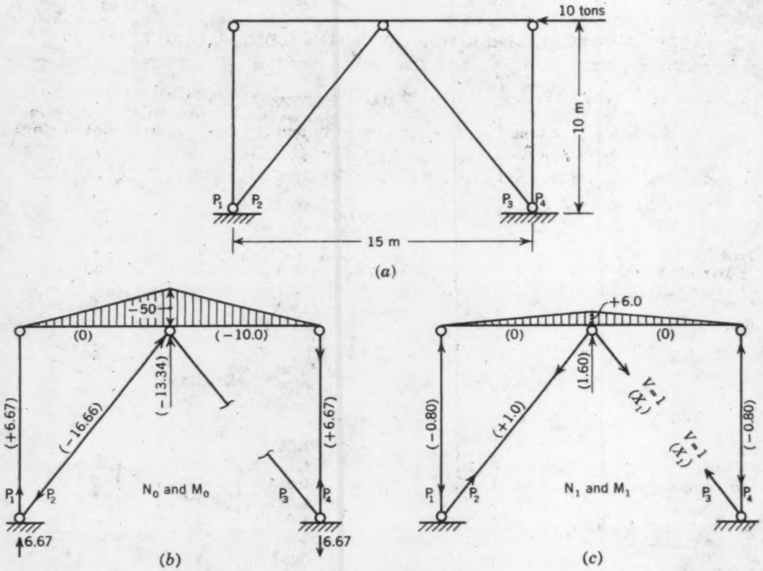

10 tons
10 m
P_1
P_2
P_3
P_4
15 m
(a)
−50
(0)
(−10.0)
(+6.67)
(−16.66)
(−13.34)
(+6.67)
N_0 and M_0
6.67
6.67
(b)
+6.0
(0)
(0)
(−0.80)
(+1.0)
(1.60)
V=1 (X_1)
V=1 (X_1)
(−0.80)
N_1 and M_1
(c)

FIG. 13

Example 6.—Using the more exact method, the forces in the piles in Example 2 (Fig. 6) will be determined (Fig. 13).

Assume a deck cross section of 0.4 m by 1.0 m (area = 0.4 sq m and I = 0.033 m^4) and a pile cross section of 0.4 m by 0.4 m (area = 0.16 sq m and I = 0.00213 m^4); thus $I_{deck} = 15\ I_{pile}$.

The axial force in the pile can be determined from

$$\delta_{10} + X_1\ \delta_{11} = -\ X_1 \frac{L_3}{A_3\ E} \qquad \text{. . . . . (45)}$$

for which

$$\delta_{10} = \sum \frac{M_0\ M_1}{E\ I}\ \Delta L + \sum \frac{N_0\ N_1}{E\ I}\ \Delta L$$

$$= \frac{1}{E}\left[-2\ \frac{(50)(7.5)}{2}\ \ 4\ \ \frac{1}{0.033}\right]$$

$$+ \frac{1}{E}\left[-\ 2\ \frac{(6.67)(0.80)(10)}{0.16} - \frac{(16.66)(1)(12.5)}{0.16}\right]$$

$$= -\ 0.023734\ \text{m/E}$$

$$\delta_{11} = \sum \frac{M_1^{\,2}}{E\,I}\ \Delta L + \sum \frac{N_1^{\,2}}{E\,I}\ \Delta L$$

$$= \frac{1}{E}\left[2\ \frac{(6)(7.5)}{2}\quad 4\ \frac{1}{0.33}\right]$$

$$+ \frac{1}{E}\left[2\ \frac{(0.8)(0.8)(10)}{0.16} + \frac{(1)(1)(12.5)}{0.16}\right]$$

$$\simeq 0.002814 \text{ m/E}$$

and

$$\frac{L_3}{A_3\ E} = \frac{12.5}{0.16\ E} = \frac{78}{E} = 0.000039 \text{ m/E}$$

Therefore

$$-\ 0.023734 + 0.002814\ X_1 = -\ 0.000039\ X_1$$

from which

$$X_1 = 8.33 \text{ tons}$$

Thus the axial forces are as follows: For the vertical piles, P_1 and P_4, the force = 0 and for the batter piles, P_2 = -8.33 tons and P_3 = 8.33 tons. In Example 2: $P_1 = P_4 = 0$, P_2 = 8.3 tons, and P_3 = 8.3 tons.

Example 7.—The more exact solution will be used to get the forces in the piles of Example 3 due to a horizontal force of 10 tons (Fig. 14).

The deck and pile data of Example 6 is used in this example. Eq. 45 can be used to determine the axial forces. It is therefore necessary to determine

$$\delta_{10} = \frac{1}{E}\left[-\ \frac{(109)(5.75)}{2}\ 3.66\ -\ 3.5\ \frac{(109\ +\ 67.8)}{2}\ 5.48\right.$$

$$\left.-\ \frac{(67.8)(5.75)}{2}\ 3.66\right]\frac{1}{0.033} + \frac{1}{E}\Big[(-18.90)(0.953)(10)$$

$$\left.-\ (32.30)(1)(10.50) - (11.80)(0.953)(10)\ -\ \frac{1}{16}\ (10)(0.31)(3.50)\right]$$

$$= -\ 0.05576 \text{ m/E}$$

$$\delta_{11} = \frac{1}{E}\left[2\ \frac{(5.48)(5.75)}{2}\ 3.66 + (5.48)(3.5)(5.48)\right]\frac{1}{0.033}$$

$$+ \frac{1}{E}\Big[(2)(0.953)(0.953)(10) + (1)(1)(10.5)\Big]\ \frac{1}{0.16}$$

$$= 0.003425 \text{ m/E}$$

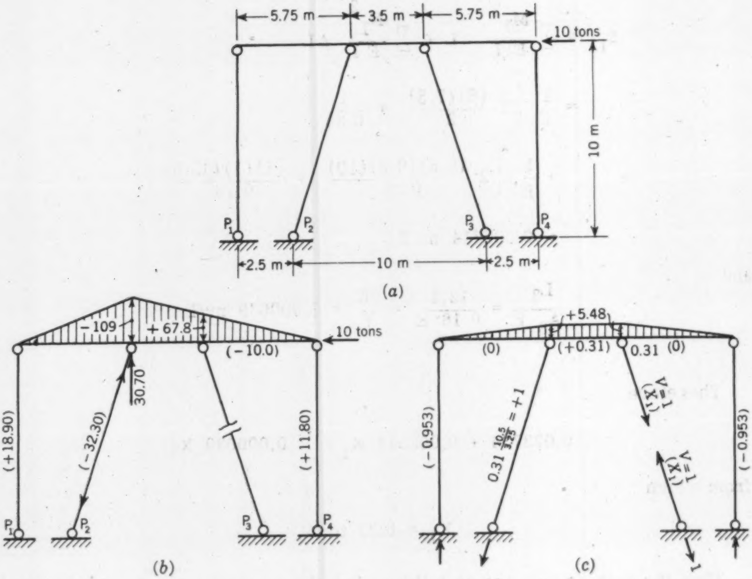

5.75 m
3.5 m
5.75 m
10 tons
10 m
P1
P2
P3
P4
2.5 m
10 m
2.5 m
(a)
−109
+67.8
10 tons
(−10.0)
30.70
(+18.90)
(−32.30)
(+11.80)
(b)
+5.48
(0)
(+0.31)
0.31
(0)
(−0.953)
0.31 10.5/3.25 = +1
V = 1 (X1)
V = 1 (X1)
(−0.953)
1
(c)

FIG. 14

and

$$\frac{L_3}{A_3\ E} = \frac{10.5}{0.16\ E}$$

Therefore

$$-\ 0.05576\ +\ 0.003425\ X_1\ =\ -\ 0.000033\ X_1$$

from which

$$X_1 = 16.1 \text{ tons}$$

Thus the axial forces are as follows: For the vertical piles, $P_1 = 3.6$ tons, and $P_4 = -3.6$ tons. For the batter piles, $P_2 = -16.2$ tons and $P_3 = 16.2$ tons. In Example 3 it was found that $P_1 = P_4 = 0$, $P_2 = -16.1$ tons, and $P_3 = 16.1$ tons.

CONCLUSIONS

Methods of solution presented herein offer simple procedures for the analysis of load distribution in any type of pier or group of piles which can be quickly applied. Difference between results obtained by approximate and exact solutions is small and thus both methods are acceptable.

APPENDIX.—REFERENCES

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 3167

SURVEYS AT COUGAR RESERVOIR

By Charles N. Oros[1]

SYNOPSIS

An outline is given of surveys made in the Willamette Valley preliminary to the special surveys at Cougar Reservoir and Dam Site. Factors which must be considered in large scale photogrammetric mapping of precipitous high relief areas are presented and how these problems were resolved using projector type stereoplotting equipment.

This paper presents a chronological story of the surveys at Cougar Reservoir located about 50 miles east of Eugene, Oreg., on the west slope of the Cascade Range. It emphasizes the part photogrammetry contributed to the project. Surveying is mentally pictured as something accomplished in the field, out in the open, using tape, transit, level and axe; it is, although to a much lesser degree, for a photogrammetric survey. Without the basic data gathered in the field and transferred to the photographs as picture-point control, an accurate photogrammetric map cannot be made.

During the years 1935 and 1936 extensive surveys were made in the Willamette River Basin. Many monuments were set, over which traverse and level lines were run. Aerial photographs were taken of all the cultivated areas and reservoir sites. Plane table surveys and geology investigations were made of proposed dam sites and reservoirs. From this data reports were written and plans formulated to build dams that would control the recurring winter and

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spring floods, improve navigation and irrigation through regulation of stream flow, and develop electric power. Since the initial report there have been further site investigations and reports, design and construction surveys, as well as actual construction of dams.

Wherever possible the surveys were tied and adjusted to United States Coast and Geodetic Survey triangulation and levels. Without exception this was possible for the level lines, but the traverse lines fared poorly. Only those in the main valley of the Willamette River were tied and computed on either geodetic or Oregon State coordinates. The reservoirs were in areas that would have required considerable time and expense to have tied them into the then existing triangulation network. However, during later years, the United States Coast and Geodetic Survey, the United States Forest Service, the United States Geological Survey and the United States Army, Corps of Engineers, have extended the triangulation network so that all of the constructed dams and most of the sites now under investigation are tied into a common datum - the Oregon State Coordinate System.

To date, all surveys, plans, and designs at the Cougar Reservoir site have been worked on a local coordinate system. During 1957 a United States Geological Survey triangulation party made a geodetic tie to the main traverse while establishing control for their quadrangle mapping. Consequently, in time, all of the Cougar Reservoir data will be converted to Oregon State Coordinates.

In 1945 the reservoir area was in virgin timber along the South Fork of the McKenzie River, accessible only by a Forest Service Road adjacent to the river. During that year a triangulation monument was placed on each abutment as reference points for future surveys. They were tied into a short traverse run at that time, which was computed on local coordinates. Later, reconstruction of the road destroyed all but one of the traverse points.

During 1953, the Cougar Reservoir became an active project. Two-in. iron pipes with railroad spikes cemented therein were set along the Forest Service Road throughout the length of the proposed reservoir. From the triangulation monuments on each abutment a traverse was run through the new monuments and computed on the original local coordinate system. Downstream about 3 miles from the dam site there is a United States Coast and Geodetic Survey first order level line along the McKenzie River Highway. From this line a single run level line was run to establish elevations on the monuments set along the road. These monuments, on 1929 datum and local coordinates, became the basic control for all subsequent surveys for the planning, geology, design, relocation, cadastral and construction phases of the program.

The first bit of topographic mapping was the dam site area. Several plane table sheets at a scale of 100 ft per in. with 2 ft and 5 ft contour intervals were completed during the summer of 1953. At the same time aerial mapping photography was taken of the reservoir area with an $8\frac{1}{4}$ -in. camera. Picture points were selected and described on a set of photographs which were then sent to the field for a contract survey crew to make proper ties. Horizontal picture point control was chosen along the Forest Service Road, with the points as close as possible to the traverse-level line monuments. Finding suitable places for vertical picture-point control was very difficult because the area was covered with timber up to 160 ft or more in height. Occasionally a small open area would appear in the timber or a stream junction could be determined on a photograph. Elevations were obtained at these points as well as along the Forest Service road that meandered somewhat through the center of the flight strip.

Photogrammetrically speaking, the multiplex extensions made from the foregoing control left much to be desired because there were not enough vertical picture points per model. Also, absolute readings could not be taken in many cases, and pass points in tall, dense timber are uncertain. These tend to leave the stereoscopic instrument operator with a feeling of uncertainty. Nevertheless a topographic map of the reservoir area was completed in 1955 at a scale of 1:9600 with a 20-ft contour interval.

The question may be raised: How can a contour be delineated correct to plus or minus 10 ft when it is not possible to see the ground because of timber 160 ft or more in height? The answer is that it cannot be done. Neither can it be done by any field method at anywhere near a comparable cost. To obtain the 8 horizontal and 45 vertical picture points requested to prepare said reservoir map of 14 sq miles, the field costs were more than the total photogrammetric costs, including contract payments for flying, making mosaics, drafting of dam site topography and other incidentals.

In spite of the unfavorable conditions for preparing this topographic map it is surprising how good it actually is. During 1956, preliminary lines for road relocation were run on both sides of the proposed reservoir at about elevation 1,700 ft mean sea level, just above the full pool elevation. Every seventh station for approximately 8 miles along the right bank and 2-1/2 miles along the left bank was plotted, without favoritism. In fact, in order to check the stereoplotting work, the right bank section was chosen to go through the area of greatest uncertainty. Of the 60 points checked without benefit of horizontal movement as is provided in map accuracy specifications, 36, or 60%, were correct within one contour interval — that is plus or minus 20 ft. Similarly, 15, or 25% were plus or minus 20 ft to 50 ft in error and the remaining 9, or 15%, ranged up to 150 ft in error. The maximum horizontal displacement of any contour at a point checked was 0.15 in. at the map scale of 800 ft per in., with only 10, or 17%, displaced over 0.10 in. and 25, or 40%, under 1/30 in.

The results cited are normal at this office. At Hills Creek Reservoir, on the Middle Fork of the Willamette River, with similar terrain, ground cover, control, mapping and checking, the accuracy results were almost identical for a 1:12,000 scale map. Also, a multiplex set-up indicated an error of approximately 100 ft at a certain point in the basic traverse. A field check verified that one chain length had been dropped in the original work.

At this point a word of caution, or perhaps advice or suggestion is appropriate. It is believed that the preceding results of the photogrammetric surveys are very satisfactory. However, it is not possible to purchase a stereoplotting instrument, pick up a man to operate this type of precision equipment, and then expect to obtain similar satisfactory results. Each person concerned with these phases of photogrammetric mapping had fifteen or more years of experience in his specialty. This experience, combined with concientiousness, is essential. So, also, is the right kind of aerial photography and picture point control. Even then, absolute accuracy is not obtainable under the foregoing conditions. Nor is it obtainable by ground surveys, unless a tremendous amount of money is expended.

The dam site and reservoir maps described were used for reports, planning, and design until larger scale maps of greater accuracy were required. For road relocation, cross sections were taken from the before mentioned preliminary lines. The usual process of plotting notes, projecting an alignment, computing cuts and fills to establish a line and staking the located line for construction sufficed for most of the road relocation.

In open or scattered brush country, 5-ft contour interval 200 ft per in. maps are stereoplotted. These maps are used to project the proposed road and/or railroad relocation alignment, thereby eliminating the field work of running a preliminary line and taking cross-sections. That reduces overall costs considerably, because only the located line needs to be staked.

By 1956 the Cougar Dam Site area had been cleared. As design progressed, serious discrepancies appeared in the 100 ft per in. plane table topography. This map, made before any clearing of the site, was a compromise between accuracy, cost, and safety. Not only were insufficient rod readings taken because of the dense brush and timber, but much of the area is inaccessible. Consequently, during the latter part of 1956, a major portion of the dam site was mapped photogrammetrically to replace the original survey.

In the spring of 1957 stripping of overburden was started. This operation was not necessary in the rock outcrop area of near vertical and overhanging cliffs at each abutment, but excavation quantities for the spillway, road relocation and main dam would have to be determined and paid for. The thought of accidents to surveymen hanging by ropes in order to take cross sections created the following questions "Could it be done photogrammetrically, at 40 ft to the in. with elevations correct to 1 ft?"

Here was a problem, a challenge to photogrammetry, that was accepted with reservations and solved with compromises and unorthodox procedures. Some of the problems and their solution are given in the following paragraphs.

The river elevation is approximately 1,280 ft and the clearing line above the abutments around 2,200 ft, mean sea level, a difference of 900 ft. At 40 ft per in. the vertical movement of the tracing table must be 22.5 in. Optimum projection distance for the plotting equipment is 21 in. plus or minus 4 in. This can be stretched to plus or minus 5 in. which would allow a 10 in. vertical throw, but 22.5 is needed.

A map scale of 40 ft per in. required a negative scale of 136 ft per in., or 1:1630. Flight altitude above mean datum using a 6-in. lens would be 815 ft. Mean datum of 1,750 plus 815 is 2,565 ft mean sea level or only 365 ft above the cleared areas at the timber cutting line. Maximum forward lap that can be accommodated in the stereoplotting equipment is 71%. Net forward movement of the airplane for 71% overlap on a 9-in. negative at a scale of 136 ft per in. is 355 ft. It is then determined that 355 ft is 50% overlap for a negative scale of 79 ft per in. Flight altitude for that scale is 475 ft above ground. Therefore elevation 2,090 ft mean sea level, 2,565 minus 475, would be the upper limit of mapping photography coverage. That is not high enough. Also, to obtain exposures 355 ft apart, they would have to be taken at 4-sec intervals from a plane flying at 60 mph. Normally, precision aerial mapping cameras do not operate that fast and 60 mph is not a safe flying speed when the timbered hills are just off the wing tips.

From the foregoing it can be seen that every phase of the operation would be right at or beyond the limit of equipment capability, with no tolerances available. Obviously another approach to the problem should be taken.

Just twenty years ago it was considered that the solution just could not be done. However, in a slightly different approach, the author's office has been doing it occasionally for the past fifteen years. The solution was to take the photography with an 8-1/4-in. camera and use it in a 6-in. projector.

The difference in elevation in a single model is approximately 900 ft. With a vertical plotting scale of 70 ft per in., the vertical range of the floating mark must be 900 divided by 70 or 13 in. This is stretching the 10-in. range of sharp focus, but it remains satisfactorily workable. With matched equipment, that is 6-in. photography used in a 6-in. projector, the horizontal and vertical scales are identical. But using unmatched equipment, that is 8-1/4-in. photography in a 6-in. projector, the vertical scale is 69.44 ft per in. and the horizontal scale is 50-ft per in.

So, in addition to 8-1/4 in. photography, the horizontal map scale of 50 ft per in. was accepted. Photographic scale for a 50-ft per in. map is 170 ft per in. or 1:2040 and would require a flight altitude above datum of 1,400 ft instead of 815 ft. That allows some tolerances in the several factors previously considered. Further, although it was necessary to use the lowest areas for control in order to set up the models, it was not necessary to contour below elevation 1,500 ft mean sea level. Consequently, the mean datum was taken at 1,800 ft. Also, it was decided to work the stereoplotting equipment on the long side of the optimum projection distance. Combining all these factors would allow a little more flight elevation above ground.

Other complications were caused by the east-west direction of the axis of the dam. First, the steepest part of the abutments is on the north side, which puts that side in shadow. Second, crevices into each abutment accentuated the problem. By observation, the project engineer determined minimum shadow at the west abutment occurred at 10:15 ante meridian and for the east abutment at 2:45 post meridian. A contract to fly at those hours was consummated. Other specifications were: Flight altitude, 3,300 ft mean sea level; exposures pinpointed; two flight lines over each abutment; two passes over each line with both a 6-in. and an 8-1/4-in. camera; camera and lens passed by the United States Bureau of Standards for aerial mapping photography; and all possible precautions taken to obtain truly vertical exposures at the specified elevation.

Pinpointed exposures were requested in order to have the exposure stations at points where minimum blind spots would occur. In other words, the best perspective points for stereoscopic vision of the sheer rock walls, crevices, and overhanging cliffs were necessary.

Six-in. photography, duplicating the 8-1/4-in., was taken to insure success and for checking. Under the conditions being considered, that is, 8-1/4-in. photography being used in six-in. projectors, the aerial photography must be perfect. That means, no tip or variation in flight altitude can be tolerated. For that reason, and to further insure success, two runs over each strip were requested. Air currents in the canyon, slow speed of the plane, likelihood of incomplete vacuum, lack of overlap and missing the exposure stations were too many hazards to expect success with just one run. By the time the actual setting up and working of the models could take place, re-flights would be out of the question because the sun altitude would be too low.

Prior to the date for flying, crosses, made from two by eight-foot pieces of plyboard painted white, were placed over all the monuments in the area. There were 13 in all, tied together by triangulation and levels. After receipt of file prints, two additional horizontal and several vertical control picture-points were marked on a set of photographs. These were sent to the field for appropriate ties. Most of the vertical picture point elevations were tops of stumps, plus the elevation of the ground on the uphill side of the stump.

Except for three small areas in complete shadow or with overhang, a topographic map was made of the entire dam site construction area and portions of the road and quarry areas. A 2-ft interval was used throughout, but the horizontal scale varied from 40 to 69.44 ft per in.

The right abutment manuscript was prepared with one exposure from each flight line of the 8-1/4-in. photography. This permitted the best stereoscopic vision into the crevices. To obtain the north face of that rock outcrop, two 6-in. photographs were selected, one from each flight line. Map scales for the manuscripts were 40 and 60 ft per in., respectively. Because contours lower than 1,700 ft were not obtained in either of the above set-ups, a third pair, using 6-in. photography, was set up at a scale of 69.44 ft per in. Approximately one-half of the area contoured was duplicated in each of these three set-ups. This afforded an opportunity to cross check the work. For identical points, differences in elevation were less than a foot; for position, it was plotting ability. To compare contours drawn in the overlap area is difficult to explain, or to arrive at definite conclusions. In clearly defined areas, areas of no shadow or blind spots, a contour displacement of 0.04 to 0.02 of an in. on the map, corresponding to less than 3 ft on the ground, did happen. Generally, they crossed with maximum deviations of 0.02 in., approximately 1 ft on the ground. But even this slight drift may mean 80 ft difference in elevation, instead of 1 ft which would be one-half of the contour interval, generally considered as a criterion in accuracy specifications.

Most of the left abutment was mapped at 50 ft per in. using 8-1/4-in. photography. Due to slight variation in flight altitude of the 8-1/4-in. photography, the quarry area to the south and road relocation area to the north had to be worked with 6-in. photography at 62.5 ft per in. Here again there were overlapping areas which provided a check on the stereoplotting work: different models, different or additional control, different cameras, and different flight strips. Results of these checks were the same as those for the right abutment. Also, the P2-line for the west side road relocation was plotted after completion of the map. Maximum discrepancy of the six points which fell in the contoured area was 1 ft.

The various manuscripts described were brought to a 50-ft per in. scale for tracing. This set is the final or basic topographic map for the dam site. From it, excavation quantities will be computed for contract payments. It also was used for projecting the road through the left abutment area. Reduced to 100 ft per in., a positive transparency was prepared with only the 50-ft contours retained above the 1,300-ft elevation. This became the basic topographic dam site map on which appropriate data were superimposed for each of several contractual features.

Many models were tried that came very close to setting up but were not acceptable because of a small amount of parallax someplace in the model; an elevation difference of 1/2 ft or more at a vertical picture-point control station; or a one-foot or more displacement at a triangulation point. Hours, even days, were spent bringing the models used into absolute orientation, and parallax free. In addition, the area of each model used was carefully correlated with the area of least distortion for the lens of the taking camera as determined by the United States Bureau of Standards test for that lens. Whenever these precautions were overlooked, horizontal displacement of contours became excessive—in a few instances as much as a tenth of an in. at the map scale.

In conclusion, the foregoing narration has dealt with the major surveys of the Cougar Reservoir site on the South Fork of the McKenzie River east of Eugene, Oreg. There were other surveys at the site, construction surveys will continue throughout the construction period, and there will be various surveys after construction is completed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3168

SDR SURVEY FOR PROPOSED CHESAPEAKE BAY CROSSING

By Walter C. Beckmann,[1] Charles L. Drake,[2] and George H. Sutton[3]

SYNOPSIS

The application of a wide band, spark source echo sounder, the Sub-bottom Depth Recorder (SDR) to foundation studies for the proposed Chesapeake Bay Highway Crossing from Little Creek to Cape Charles, Virginia, is described in detail. Reflection horizons are correlated with varying properties of the sediments as revealed in test borings.

INTRODUCTION

Only in recent years have geophysical techniques been used to aid in the solution of problems encountered in foundation engineering. This is due to the novelty of the techniques, to the concentration of geophysical efforts in other fields, and to the unfamiliarity of the foundation engineer with geophysical methods. The application of many of the tools developed for geophysics can offer not only a saving of time and effort to the foundation engineer but furnish him with additional information not made available by standard practices.

An excellent example of this is a recently developed sonic reflection equipment, the Sub-bottom Depth Recorder (SDR). This equipment is similar to

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standard echo sounding equipment used to determine water depth. Utilizing high powered, low frequency sound sources and variable pass ban detection, it permits the mapping of not only the bottom topography but the structure beneath the bottom as well.

Using the SDR, it has been possible to map sub-bottom horizons at depths exceeding 1,600 ft in regions with high acoustic impedance contrasts. Of equal importance is the fact that by judicious selection of pass band frequencies very small changes in the lithology of the sedimentary column may often be distinguished and clearly mapped.

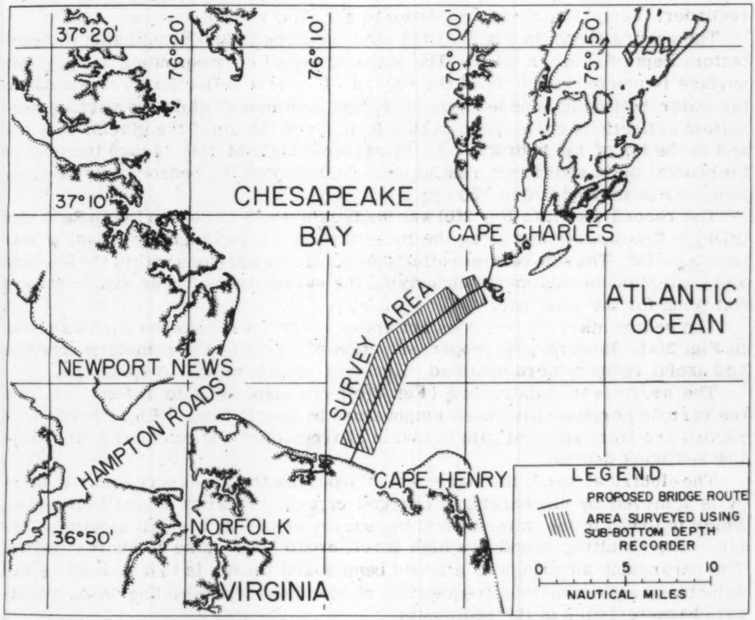
37°20'
76°20'
76°10'
76°00'
75°50'
37°10'
CHESAPEAKE
BAY
CAPE CHARLES
ATLANTIC
OCEAN
SURVEY AREA
NEWPORT NEWS
HAMPTON ROADS
CAPE HENRY
36°50'
NORFOLK
VIRGINIA
LEGEND
PROPOSED BRIDGE ROUTE
AREA SURVEYED USING SUB-BOTTOM DEPTH RECORDER
0
5
10
NAUTICAL MILES

FIG. 1.—LOCATION OF SURVEY AREA

During the latter part of January 1958, a SDR was used to survey the area of the proposed Chesapeake Bay Highway Crossing between Cape Charles and Little Creek, Virginia, shown in Fig. 1. The survey area (shaded) parallels the highway crossing route extending a mile on either side. The area was in open water and subject to weather and seas entering Chesapeake Bay from the Atlantic Ocean. In this shaded area a one half mile grid was surveyed covering over 100 miles of track. Lines of particular interest were surveyed several times using various filter and gain parameters. The grid lines are shown subsequently in Fig. 4. Although there were no extensive interfaces with high impedance contrasts in the survey area, it was possible to map a number of horizons within the relatively homogeneous sediments.

DESCRIPTION OF EQUIPMENT

The SDR is similar to a sonar echo sounder. The model[4] used in this survey utilizes a broad band, low frequency sound produced in the water by an electrical spark discharge. The energy reflected from the bottom and sub-bottom horizons is detected by a towed pressure-sensitive hydrophone. The signal is suitably amplified, electronically filtered, and displayed on a chart recorder. Sample records are shown in Fig. 2(a) and (b).

The record shown in Fig. 2(a) was made on Long Island Sound using the Sub-bottom Depth Recorder (SDR). The outgoing signal corresponding to the water surface is printed at the top of the record. The first reflection corresponds to the water bottom)composed here of recent sediments) while the various sub-bottom reflections correspond in this instance to the top of the glacial deposits and to the top of the bedrock. At the extreme right of this record the edge of the coastal plain sediments, resting unconformably on the bedrock, can be seen. Receiver pass band 250 to 750 cps.

The record shown in Fig. 2(b) was made using the Sub-bottom Depth Recorder (SDR) in Chesapeake Bay along the route of the proposed highway crossing near bore hole D2. The sub-bottom reflections are from horizons within the Miocene and post-Miocene sediments underlying the survey area. These sediments are flat lying for the most part.

The sedimentary column does not undergo any great changes such as shown in Fig. 2(a). However, by proper selection of equipment parameters readable and useful records were obtained. Receiver pass band 400 to 4,000 cps.

The arrivals on this record (Fig. 2(b)) corresponding to reflections from the various horizons have been emphasized by pencil lines. Light portions of record are from reduced gain to obtain sharper observation of water and shallow sediment depths.

The electrical spark discharge which produces the necessary accustic energy is achieved by discharging a charged capacitor across a small spark gap, which is towed in the water behind the survey vessel by a small electrical cable. The resulting sound is a high level, broad band pulse of short duration. The paramount advantage of a broad band sound source is that sediments will selectively reflect various frequencies of sound energy depending on the physical characteristics of the sediments.

The ability, through the use of electronical filtering, to select the optimum frequency band of reflected signal permits one to adapt the SDR to different sub-bottom conditions. The higher frequencies were used in this area to produce a cleaner, more detailed record since the physical changes in the sediments were small and in many cases the thicknesses involved were slight.

The ratio of incident to reflected energy for a given horizon depends chiefly on the acoustic impedance change across the interface and on the ratio of the wave length of the sound used to the dimensions involved. In general the lower frequencies penetrate deeper into unconsolidated sediments and the higher frequencies are preferable for studies of fine structure. In an area such as Long

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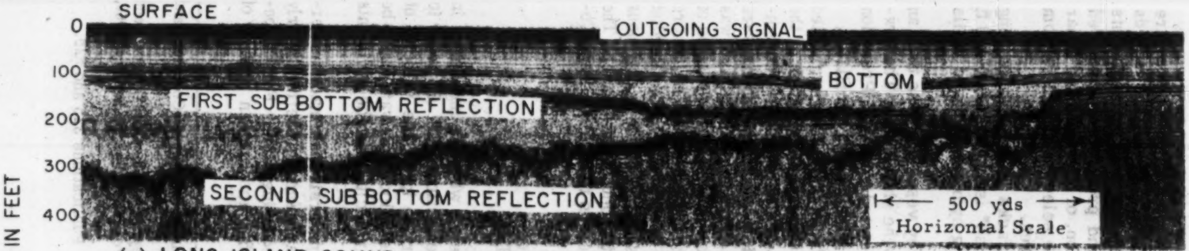
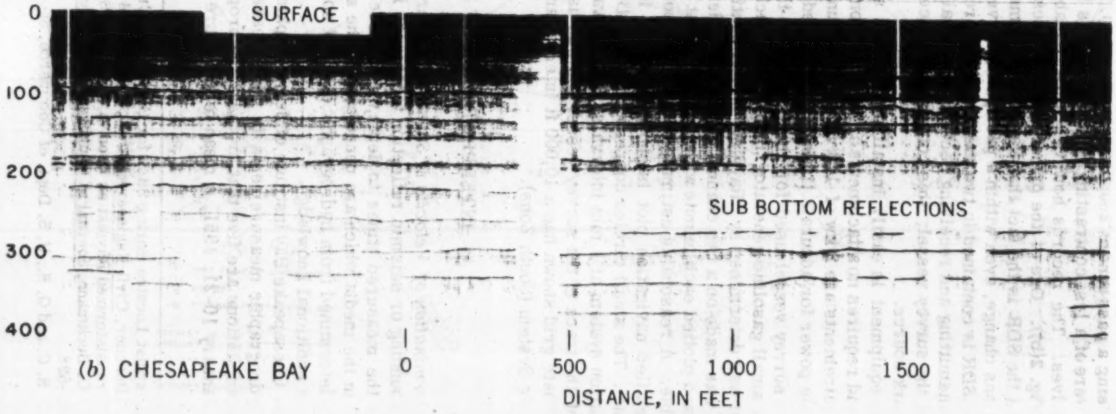
[4] "Sub-Bottom Depth Recorder," by Walter C. Beckmann, Archie C. Roberts, and Bernard Luskin, Geophysics, Vol. 24, No. 4, October, 1959, pp. 749-760.



(*a*) LONG ISLAND SOUND



(*b*) CHESAPEAKE BAY

FIG. 2.—TYPICAL SDR REFLECTION RECORDS

Island Sound where the sub-bottom horizons have large impedance contrasts and where the thicknesses of these horizons are large, use of high frequencies is not advantageous, and one can achieve a better signal to noise ratio by utilizing lower frequencies. In fact the highest quality records from this area are made using a pass band of 250-750 cps (Fig. 2(a)). In the survey area the records made were of a less contrasting nature since this is the case with the sediments themselves. The records however were readable and provided the required data (Fig. 2(b)). One of the principal advantages, then, of a broad band sonar such as the SDR is the fact that the pass band may be selected as sub-bottom conditions change, even within a particular survey area.

The SDR is contained in two standard 19 in. relay racks 40 in. high, housing the transmitting and receiving sections. A small "fish" is towed about 150 ft behind the survey vessel; this contains the receiving hydrophones and trails the spark source.

The equipment is easily installed aboard a survey vessel in less than an hour and requires no attachments or modifications to the ship's hull. The power requirements are 2 kw of 115 vac from an unregulated source. Precision 60 cycle power for accurate timing is controlled by a tuning fork oscillator.

The survey vessel used for the survey was the 70 ft yacht, "Robin," fitted with a small gasoline generator mounted on deck. Geographic position of the vessel was determined by "Raydist."[5,6] A continuous plot of Raydist coordinates was made on a two channel Brush Recorder. In addition, positions were read and plotted each minute with the result that excellent position control was possible. A reasonable estimate of error of position is ± 30 ft. A portion of the detailed navigation plot is shown in Fig. 3 in the region of the Baltimore Channel. The small circles correspond to positions determined by the Raydist navigation system at 1 min intervals. Raydist was used to produce a continuous plot of the track of the survey vessel and was used to control the vessel. The coordinate grid shown has a 10,000 ft interval and refers to Virginia State Coordinate System (South Zone).[7]

INTERPRETATION OF DATA

Determination of Velocity of Sound.—The parameter which is measured in echo sounding or seismic reflection techniques is reflection time. In order to relate the measured times to depth, it is necessary to know the velocity of sound in the media which are penetrated. The sound velocity in water may be easily determined from hydrographic data but sound velocities in the sediments require additional knowledge.

The Chesapeake Bay Institute of Johns Hopkins University has made numerous hydrographic measurements in Chesapeake Bay. Among their regularly occupied stations are five that are near the proposed bridge line. For the period January 10-21, 1951, a mean temperature of 5 C and a mean salinity of

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[5] "Raydist Locates Boring Sites for Chesapeake Crossing," by Allen L. Comstock and P. Z. Michener, Civil Engineering, July, 1959, pp. 54-57.

[6] "Transistorized Raydist as Used in Geological Surveys," by Charles L. Drake and Walter C. Beckmann, Journal of Geophysical Research, Vol. 65, No. 2, February, 1960, pp. 525-528.

[7] U. S. C. and G. S., U. S. Dept. of Commerce, Special Publications 325 and 329.

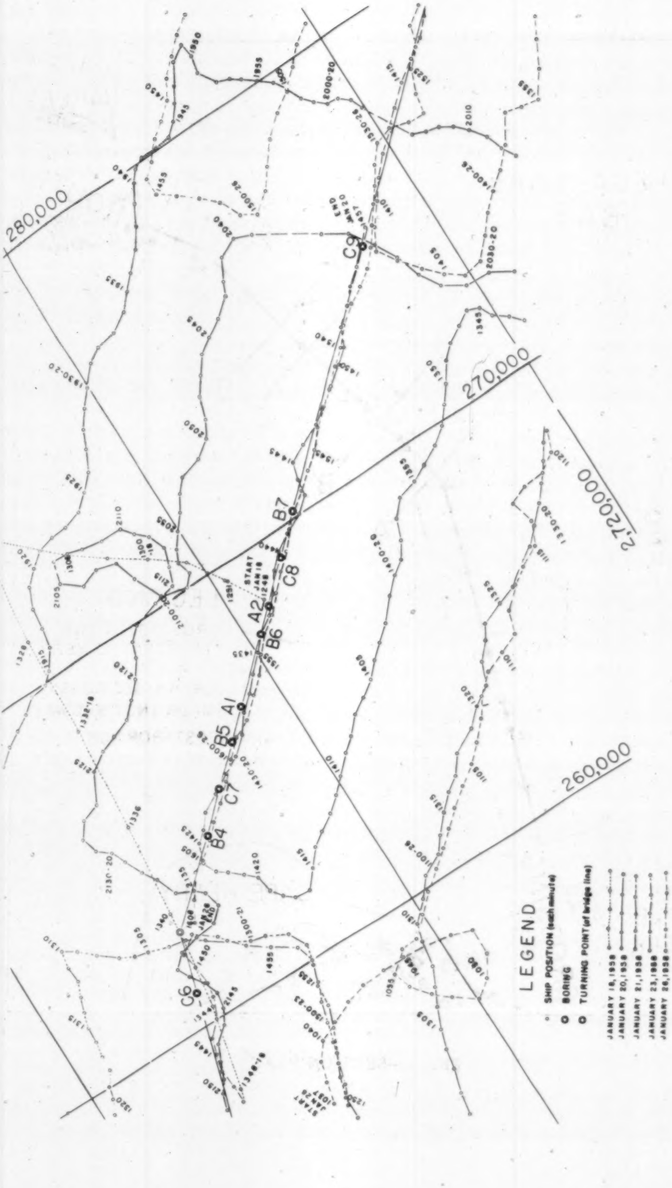
280,000
270,000
260,000
2,720,000
C9
B7
C8
A2
B6
A1
B5
C7
B4
C6
START JAN 18 1248
START JAN 26 1037
LEGEND
SHIP POSITION (each minute)
BORING
TURNING POINT (of bridge line)
JANUARY 18, 1958
JANUARY 20, 1958
JANUARY 21, 1958
JANUARY 23, 1958
JANUARY 26, 1958

FIG. 3.—DETAILED NAVIGATION

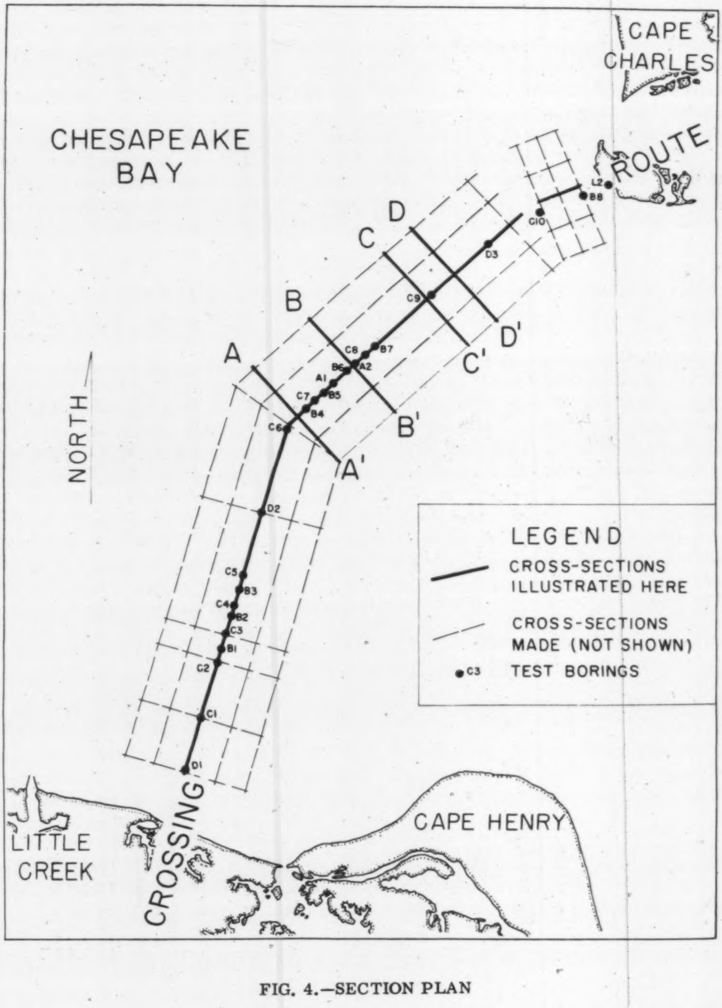
CAPE CHARLES
CHESAPEAKE BAY
ROUTE
CROSSING
NORTH
A
A'
B
B'
C
C'
D
D'
L2
B8
C10
D3
C9
B7
C8
B6
A2
A1
B5
C7
B4
C6
D2
C5
B3
C4
B2
C3
B1
C2
C1
D1
LEGEND
CROSS-SECTIONS ILLUSTRATED HERE
CROSS-SECTIONS MADE (NOT SHOWN)
C3 TEST BORINGS
LITTLE CREEK
CAPE HENRY

FIG. 4.—SECTION PLAN

28°/00 were determined.[8] The resultant computed sound velocity[9] is 4,800 ft per sec.

The variation of velocity with depth in the coastal plain sediments has been studied by J. E. Nafe and C. L. Drake[10] and a mean gradient has been determined. Using this gradient, a mean velocity of 1.2 times the velocity of sound in water was determined for the upper 400 ft of the sedimentary column. This value was used for the preliminary reading of the records.

To determine the value of sound velocity to be used for the final interpretation, comparison was made with the data. The relation between seismic compressional wave velocity and other properties (porosity, grain size, sorting, mineral content) has been studies by G. H. Sutton, H. Berckhemer and Nafe.[11] This work was carried to higher velocity values by Nafe and Drake.[10] For the type of sediments encountered in the area of the proposed highway crossing and for the porosity range indicated, porosity is the major factor contributing to velocity changes.

Velocities were computed using the velocity-porosity relationship of Nafe and Drake,[10] from porosity data of samples taken from the test borings furnished by Moran, Proctor, Mueser, and Rutledge.[12] The results show a mean value closer to 1.3 times the velocity of sound in water. This difference is due partly to the low velocity of sound in the cold water, partly to unloading of the sedimentary column by erosion, and partly to selective sampling. The first factor will cause an increase in the velocity difference if, due to poor thermal conductivity, the water is appreciably colder than the underlying sediment.[13] The second factor will be significant if the sediments were once buried to greater depths and have been exposed subsequently by erosion. In these circumstances, the velocity tends to decrease more rapidly than the porosity increases. Lastly, the water content was not measured on those high porosity samples from which it settled out. Inclusion of these would have lowered the mean velocity computed from the porosity measurements. Because of these factors the value 1.2 times velocity in water used in the preliminary readings, was also used for the final interpretation.

In addition to the velocity-porosity comparison, the computed reflection depths were compared with available boring information. The prominent reflecting horizons correlated with changes in lithology recorded on the drilling logs, confirming the choice of sediment velocity.

Accuracy of Depth Determination.—The scale of the SDR records is such that 1 ft of water depth is represented by 0.0144 in. and 1 ft of sediment depth by 0.0120 in. The reading error, as such, can probably be held to about 1 ft.

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[8] Chesapeake Bay Inst., The Johns Hopkins Univ., Data report, Cruise VIII, Jan. 10 to Jan. 23, 1951, Report No. 6.

[9] "Tables of the Velocity of Sound in Pure Water and Sea Water for use in Echo-Ranging and Sound Ranging," by D. F. Matthews, Publication H. D. 282, Hydrographic Dept., Admiralty, London, Second Edition, 1939.

[10] "Variation with Depth in Shallow and Deep Water Marine Sediments of Porosity, Density and the Velocities of Compressional and Shear Waves," by J. E. Nafe and C. L. Drake, Geophysics, Vol. 22, July, 1957, pp. 523-552.

[11] "Physical Analysis of Deep Sea Sediments," by G. H. Sutton, H. Berckhemer, and J. E. Nafe, Geophysics, Vol. 22, October, 1957, pp. 779-812.

[12] "Chesapeake Bay Highway Crossing," by Moran, Proctor, Mueser, and Rutledge Sverdrup and Parcel, Cons. Engrs., Foundation Study, Unpublished report, 1958.

[13] "Sound Velocity vs. Temperature in Water-Saturated Sediments," by G. Shumway, Geophysics, Vol. 23, July, 1958, pp. 494-505.

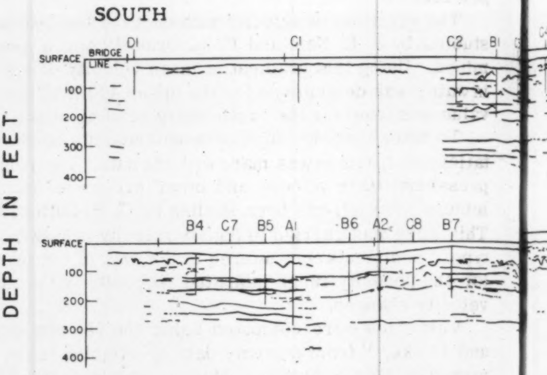


FIG. 5.—CROSS SECTION ALONG

Other errors arise, however, from the low frequencies used. In order to penetrate to any depth in the sediments, it is necessary to use rather low frequencies. Echo sounders, to measure water depths, may use frequencies between 12 and 200 kilocycles; standard seismic reflection prospecting techniques use frequencies ranging from 20 to 500 cycles. For work of the type called for by this survey, frequencies between these two ranges are used.

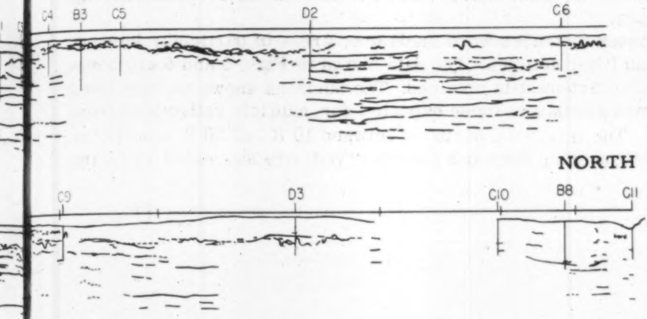
With a lowering of signal frequency there results a corresponding decrease in definition. A 200 kilocycle echo sounder uses wavelengths of 0.025 ft while a 400 cycle arrival on a sub-bottom sounder has a wavelength of 12 ft. In the latter case, the indicated depth to a reflecting horizon could be as much as 6 ft too great. The measured depths, furthermore, are somewhat sensitive to the gain setting of the receiving circuit. When higher gain is used, emergent signals are recorded at earlier times with possible reductions in apparent depth up to 6-8 ft.

Assuming that the average vertical velocity chosen was correct, and since the important sources of error tend to increase the indicated depth, a reasonable estimate of the error in the depth determination of a horizon is +10 to -2 ft. As previously indicated, the sediment velocity used has been determined to within about ± 5%. Using this velocity, the prominent reflection horizons agree with the bore hole data. Errors in the velocity used will, in general, produce errors in depth proportional to depth.

PRESENTATION OF DATA

No attempt has been made to present the multitude of SDR data which was obtained. Only examples of the type of data furnished by the SDR survey are included here. The portion of the survey in the region of the Baltimore Channel along the proposed route is considered as a particular example.

One of the principal reasons for conducting the SDR survey was to establish whether or not a buried, ancient channel of the Susquehanna River crossed the proposed bridge line. This possibility had been suggested by earlier geological



PROPOSED HIGHWAY CROSSING ROUTE.

studies in the area.[14] The widely spaced preliminary borings could easily have left such a channel undetected and its presence could alter considerably the foundation considerations.

Over a large part of the area surveyed many reflecting horizons were found to depths of about 350 ft. In general these were flat lying or had small dips. In certain areas the upper part of the section was complicated by minor channeling and other depositional or erosional processes at shallow depths. The most persistent horizons were at 150-160 ft and 270-310 ft. From Fig. 7 it can be seen that in the Baltimore channel area these can be correlated with layers in the boreholes which were difficult to penetrate and/or were of low porosity. The persistence of the upper of these two layers along the entire crossing line (Fig. 5) and on the sections on either side of the bridge line makes it extremely doubtful that the old channel of the Susquehanna could pass through the area that now forms the mouth of the Chesapeake Bay.

The total thickness of the Miocene in this area[15] is on the order of 600 ft. Since the deepest reflections received during the survey were from horizons about 350 ft deep, it must be concluded that the data are confined to the Miocene and the varying amounts of younger sediment above it.

Fig. 4 shows the sections made from the SDR data. The dashed lines indicate the longitudinal and transverse cross sections that were made using the SDR data, and the heavy lines indicate the cross sections which are shown in Figs. 5 and 6. The section along the crossing route is shown in Fig. 5. It represents a composite of records made at different receiver gain and filter settings which were adjusted to give maximum information about the various layers. The letter-numbers correspond to the position of the test borings. (The vertical line indicates the maximum depth of the bore hole.) The area between borings D3 and C10 were no bottom reflection is shown was not surveyed because of shoal water.

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[14] "Submerged River System of Chesapeake Bay," by J. T. Hack, Bulletin, Geol. Soc. of America, Vol. 68, 1957, pp. 817-830.

[15] "Subsurface Stratigraphy of the Atlantic Coastal Plain Between New Jersey and Georgia," by H. G. Richards, Bulletin, Amer. Assoc. of Pet. Geologists, Vol. 29, 1945, pp. 885-955.

Four of the 14 transverse cross sections AA', BB', CC', and DD' are shown in Fig. 6. The cross sections selected are in the area of the Baltimore Channel. No survey track was made along those portions of the sections showing no bottom topography.

A number of passes or tracks were made over areas of particular interest at different gain and filter settings. The data shown in Figs. 5 and 6 are composite tracings of the various data obtained. The horizons shown as light lines have reflection times similar to those expected for multiple reflections from shallower layers. The irregular horizons between 10 ft and 50 ft beneath the bottom, below which no deep horizons are observed, are suspected to be the top of gas saturated sediments which have extremely high sound adsorption properties.

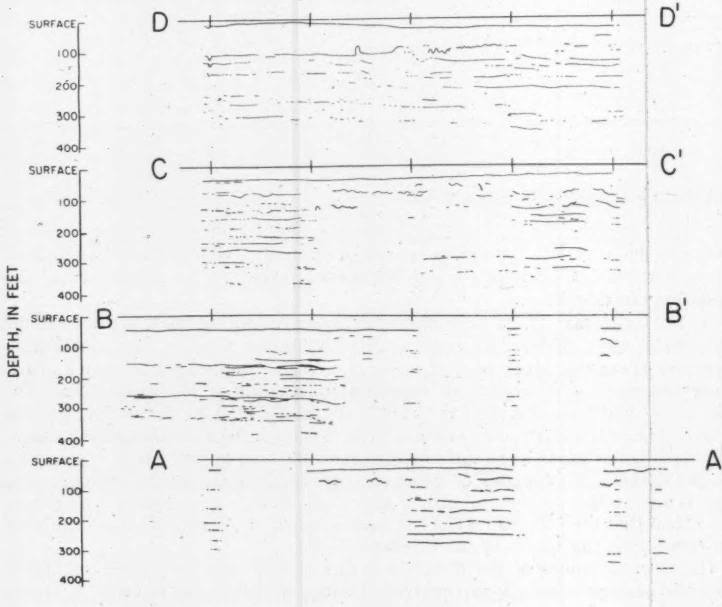

FIG. 6.—TRANSVERSE CROSS SECTIONS

The portion of the survey across the Baltimore Channel, along the bridge line, affords a good illustration of the method and of the usefulness of the SDR information. The detailed navigation for this portion is shown in Fig. 3. The positions of cross sections and tracings of records in this area can be seen in Figs. 4, 5 and 6.

Detailed results of the Baltimore Channel along the highway crossing route are presented in Fig. 7. Here tracings of the SDR reflection data[16] superimposed on the boring data[12] is shown. An excellent correlation exists between the SDR reflection horizons and the water content (porosity) of the layer (shown to the right of the borings) and the resistance to mechanical penetrations as evidenced by the number of hammer blows required to drive a split spoon sampler (shown to the left of the borings). These parameters are indicative of the acoustic impedence (density × velocity) which governs the ability of a layer to reflect sound energy.

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[16] Alpine Geophysical Assocs., Ind., Norwood, N. J., 1958, "Sub-Bottom Reflection Survey of the Proposed Chesapeake Bay Highway Crossing," Sverdrup and Parcel, Cons. Engrs., St. Louis, Mo., Unpublished report.

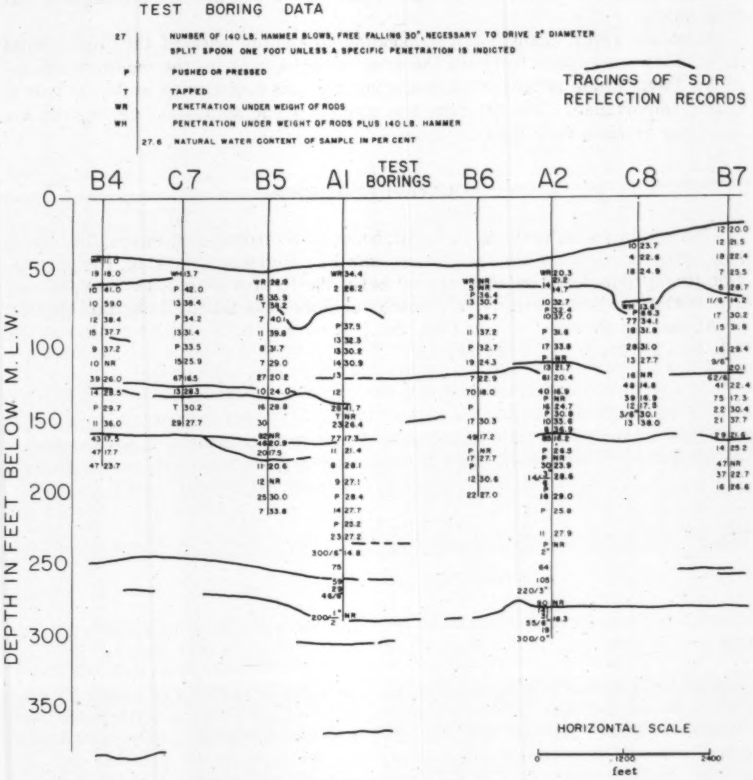

FIG. 7.—BALTIMORE CHANNEL DATA

While the SDR misses some of the minor horizons the major structure is well defined. It affords a continuous record between test borings and gives information on structure beneath the drill holes. The deeper layers were touched, in this case, by holes A1 and A2.

CONCLUSIONS

The Sub-bottom Depth Recorder (SDR) affords a method to map sub-bottom geologic structure in an economical and reliable manner. The data obtained can be correlated with test boring data and affords an excellent horizontal correlation between widely spaced test borings.

The SDR data is useful not only as an adjunct to test borings but provides a valuable guide to the most strategic planning of original or subsequent test drill holes.

A buried river channel corresponding to the old path of the Susquehanna River does not appear to cross the area covered, that is, the mouth of Chesapeake Bay. Some recent shallow channeling was found to exist but is only of minor importance. The Miocene and post-Miocene sediments of the area are flat lying or have very shallow dips.

ACKNOWLEDGMENTS

The authors wish to thank P. Z. Michener of Sverdrup and Parcel, St. Louis, Mo., project engineer for the Chesapeake Bay Highway Crossing, for permission to reproduce the data presented here; the firm of Moran, Proctor, Mueser and Rutledge, New York, N. Y., which furnished the boring data; and the firm of Alpine Geophysical Associates, Inc., Norwood, N. J., which conducted the Sub-bottom Depth Recorder Survey.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3169

SOUTHWEST PASS-MISSISSIPPI RIVER 40-FT CHANNEL

By Austin B. Smith,[1] F. ASCE

SYNOPSIS

The paper examines the heavy high water shoaling, triggered by action of the salt-water wedge, and outlines plans to obtain the authorized 40-ft deep channel connecting the Mississippi River and the Gulf via the Southwest Pass jettied channel and across the Sea Bar entrance channel. The magnitude of the annual shoaling and the associated salt-water intrusion phenomenon, prior jetty and pile dike construction, current hopper dredge requirements to maintain 35-ft-deep ship channel, the collection of new field data, and prototype studies and model tests of various improvement schemes are described. The formulated plan for providing the authorized 40-ft-deep ship channel and reducing the burdensome annual hopper dredging requirements is presented.

GENERAL DESCRIPTION

The Corps of Engineers is improving Southwest Pass to provide a dependable 40-ft-deep ship channel connecting the main stem of the Mississippi River with the Gulf of Mexico. The deep water navigation project, Mississippi River, Baton Rouge to the Gulf of Mexico, is shown in Fig. 1. The central problem is the elimination of heavy high water shoaling that forms near the mouth of Southwest Pass during each high water season. To find the best solution to this 50-yr-old problem, prototype studies and investigations and model tests of a number of possible schemes of improvement were made. These studies and tests indicated that a more sinuous channel alignment below mile 9 below head of

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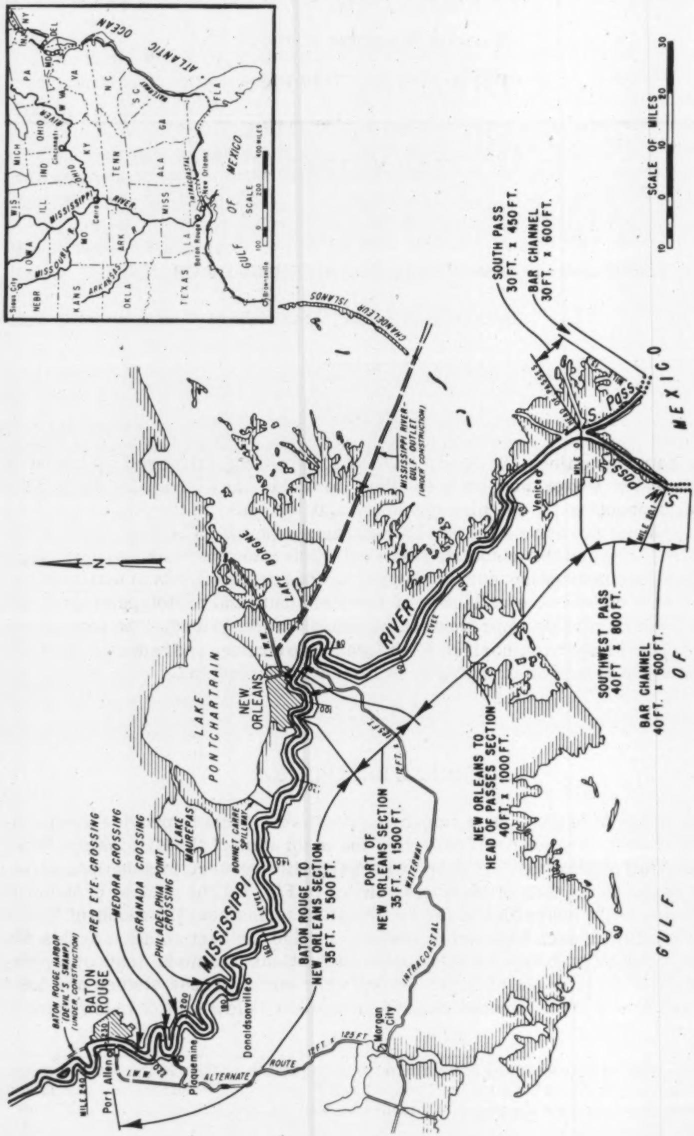
BATON ROUGE
BATON ROUGE HARBOR (DEVIL'S SWAMP) (UNDER CONSTRUCTION)
RED EYE CROSSING
MEDORA CROSSING
GRANADA CROSSING
PHILADELPHIA POINT CROSSING
MISSISSIPPI
LAKE MAUREPAS
BONNET CARRE SPILLWAY
LAKE PONTCHARTRAIN
NEW ORLEANS
LAKE BORGNE
RIVER
MISSISSIPPI RIVER-GULF OUTLET (UNDER CONSTRUCTION)
CHANDELEUR ISLANDS
Venice
S. Pass
S. W. Pass
SOUTH PASS 30 FT. x 450 FT.
BAR CHANNEL 30 FT. x 600 FT.
SOUTHWEST PASS 40 FT. x 800 FT.
BAR CHANNEL 40 FT. x 600 FT.
NEW ORLEANS TO HEAD OF PASSES SECTION 40 FT. x 1000 FT.
PORT OF NEW ORLEANS SECTION 35 FT. x 1500 FT.
BATON ROUGE TO NEW ORLEANS SECTION 35 FT. x 500 FT.
MILE 240
Port Allen
Plaquemine
Donaldsonville
I.W.W.
ALTERNATE ROUTE 12 FT. x 125 FT.
Morgan City
INTRACOASTAL WATERWAY
GULF OF MEXICO
SCALE OF MILES
10 0 10 20 30
ATLANTIC OCEAN
GULF OF MEXICO
MISSISSIPPI RIVER
OHIO RIVER
MISSOURI R.
ARKANSAS R.
Cairo
Cincinnati
Sioux City
Brownsville
Baton Rouge
New Orleans
SCALE 100 0 200 400 MILES

FIG. 1.—MISSISSIPPI RIVER, BATON ROUGE TO THE GULF OF MEXICO, NAVIGATION PROJECT

passes (BHP), and particularly in the jettied sector below Mile 17 BHP, will give additional force to the high, fresh-water flow.

MISSISSIPPI RIVER DELTA

The Mississippi River brings to its mouth sediment loads in the order of one million tons daily.[2] These land-building materials are moved mainly during the annual high water between January and July. The overlapping delta now reaches within 10 mi of the landward edge of the continental slope. Early explorers and mapmakers appropriately called the Mississippi River Delta the "Cabo de Lodo" or "Mud Cape."[3] The problem area, shown in Fig. 2, lies at the seaward end of the delta where the Mississippi River flow via Southwest Pass meets the Gulf.

Landmarks bear witness to the Mississippi River Delta's advance. The French erected the town of Old Balize at the mouth of the Northeast Pass (Pass a Loutre) in 1774. In 1834 it stood 2 mi above the mouth. In 1858 the rate of advance of new land was believed to be 96 ft a year.[4] Fig. 3 shows the seaward advance of Southwest Pass Sea Bar since the 1898 survey. The average annual advance of the 35-ft contour since 1898 has been approximately 100 ft. This rate of advance should decrease as the delta advances into deeper waters of the Gulf.

HISTORIC NAVIGATION PROBLEM

In the 400 years since European explorers discovered the mouth of the Mississippi, engineers of several nationalities have been concerned with the problem of improving the Mississippi River Passes for navigation. French, Spanish, and British engineers worked at the problem from circa 1700 to 1812. Their legacies include maps of early surveys, Bar Pilots, light houses, and river towns that are now landmarks. They named the passes for the direction in which they run.

The United States began to survey and report on the problems of deep water navigation through the passes in 1829. Countless engineers have since been concerned with this much storied problem. The records are highlighted by such names as Beauregard, Humphreys, Eads, Corthell, Ockerson, Schultz, Lipsey, Dent and Townsend, and more recently by Tyler, Ferguson, Hudson, Tiffany, Cobb, Gentilich, Kennedy and Simmons.

The problem in the 17th and 18th centuries was to find 12 to 15 ft for ships via Pass a Loutre. In the 19th century, it was to provide a 30-ft ship channel via South Pass. In the 20th century the problem was first to obtain 35-ft and now 40-ft through Southwest Pass.

The first Federal appropriation was made in 1836 and 1837 to improve Pass a Loutre. The chronicle of feeble attempts to improve the passes continued through 1875. The controversy that raged from 1871 to 1875 over whether to improve the channel of one of the passes channels by means of a jetty system or to provide a connecting channel through a canal and lock was settled in 1875 when Congress accepted Eads' "No cure, No pay" proposition. This proposition was "to secure by construction of jetties and auxiliary works and maintain for 20 years a channel 26 ft deep by 200 ft wide having through it a central depth of 30 ft without regard to width," for the sum of $8,000,000. Eads desired to improve Southwest Pass, but the Congress directed that he experiment with South Pass.[5] The impact of Eads' success was immediate and New Orleans became a world port.

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[2] "Sedimentary Framework of the Modern Mississippi Delta," by H. N. Fisk, et al., June, 1954.

[3] "Gulf of Mexico, its Origin, Waters, and Marine Life," U. S. Dept. of Interior, Washington, D. C., 1954.

[4] "Report on Physics of the Mississippi River," by Humphreys and Abbott, 1861.

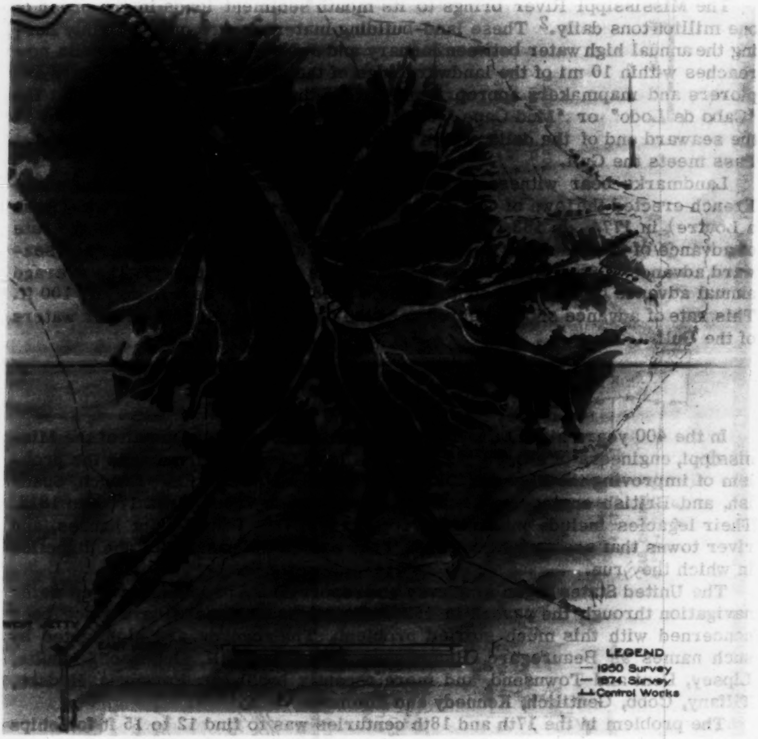

FIG. 2.—DELTA AND PASSES, MISSISSIPPI RIVER

Improvement of the navigable depths in the passes can be correlated with new concepts in channel improvement, such as the European concept of jetties that was used by Eads and Corthell from 1875 to 1879 to improve South Pass for the passage of 30-ft draft ships. It can also be correlated with the development of dependable dredging equipment, such as the modern hopper dredge Langfitt, shown in Fig. 4, now used to maintain a 35-ft channel via Southwest Pass.

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[5] "History of the Jetties," by Corthell, 1881.

PREVIOUS IMPROVEMENT OF SOUTHWEST PASS AND SEA BAR CHANNEL

New Orleans port commerce increase rapidly following Eads' successful improvement of South Pass in 1875. By 1900 a deeper and larger channel through the passes was needed to accommodate the larger and deeper draft ships bound to and from New Orleans. In 1902 the Congress authorized the project for improvement of Southwest Pass to secure a 35 ft by 1,000 ft channel, as recommended by a 1899 board, at an estimated cost of $6,000,000, with annual maintenance of $150,000. This project included construction of two jetties, sills across the major outlets and dredging. When this work was completed in 1908

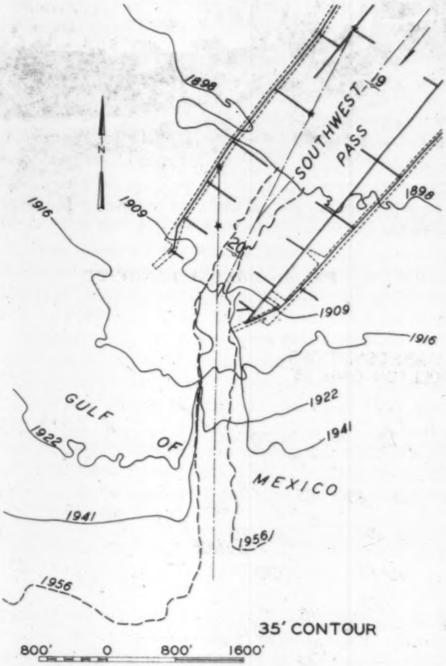
N
1898
SOUTHWEST PASS
19
1916
1909
20
1898
1909
1916
1922
GULF OF MEXICO
1922
1941
1941
1956
1956
35' CONTOUR
800' 0 800' 1600'

FIG. 3.—ADVANCE—SOUTHWEST PASS SEA BAR

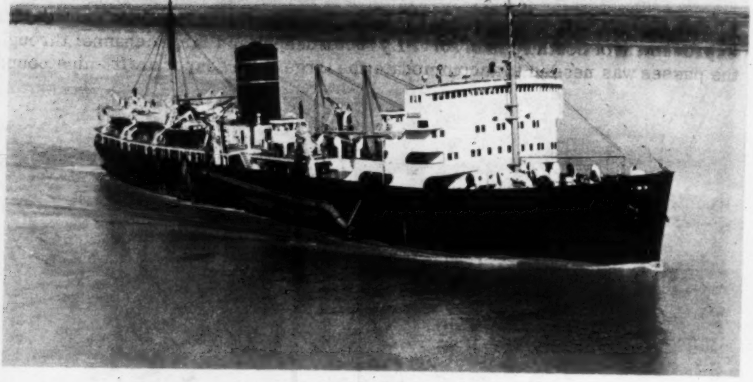

FIG. 4.—DREDGE LANGFITT

JUNE 1920 CONDITIONS
CARROLLTON GAGE 18'
N
W JETTY
E JETTY
BULKHEAD
RANGE
ENTRANCE
GULF
OF
MEXICO
60
35
12
20
20
27 21
15
23
25
28
22
16
12
16
18
20
29
26
30
35
35
54
31
32
29
26
15
14
17
20
25
27
21
25
21
23
27
20
15
16
21
15
20
15
12
11
8
35
35
20
60
100'
1000'

FIG. 5.—JUNE 1920 CONDITIONS, SOUTHWEST PASS AND SEA BAR CHANNEL

the depth over the sea bar was increased from only 9 ft to 20 ft, far short of the expected 35 ft.

From 1911 to 1912 the east and west jetties of Southwest Pass were extended seaward 2,900 ft and 3,400 ft, respectively. The project dimension of 35 ft by 1,000 ft was not obtained by the jetty extension, but the natural depth was slightly increased.

After the 1911 to 1912 extension to the jetties, spur dikes were constructed in the lower 7 mi of the pass, contracting the width between dikes to about 3,000 ft. Again the desired 35-ft deep channel was not obtained.

Between 1917 and 1921 parallel inner bulkheads were constructed in the lower 5 mi, reducing the channel width from 3,000 ft to 2,400 ft.[6] In 1924 the east and west jetties were extended approximately 1,000 ft. The outer ends were turned channelward until they intersected the projected line of the inner bulkheads. Conditions at the mouth of Southwest Pass in June, 1920 are shown on Fig. 5. The controlling depth was then 16 ft over the sea bar.

From the start of the improvements to Southwest Pass in 1904 until 1921 all dredging on the sea bar was directed toward securing a channel directly across the sea bar along the prolonged axis of the jettied channel, as shown by Fig. 5, all with very limited success. In 1921 the axis of the sea bar channel was inclined 35° to the eastward of the axis of the jettied channel. The relocated sea bar channel is shown in Fig. 6. As in the case of South Pass, 25 yr before, this action was the determining factor in the subsequent success of providing the 35-ft deep channel across the sea bar.

The jettied channel was further contracted to lessen the maintenance dredging requirements. First, as noted previously, in 1921 the channel width was reduced from 3,000 ft to 2,400 ft. Then in 1923 and 1924 the width of the lower 8 mi was further reduced to 1,750 ft between dikes and the east and west jetties were extended. This contraction, combined with 7,000,000 cu yd of dredging, resulted in a 35-ft channel in the jettied channel and across the sea bar. February, 1924 conditions are shown in Fig. 6. Between 1937 and 1939 the lower 10 mi of Southwest Pass channel were contracted to 1,420 ft between dikes. Except during years of major floods a generally satisfactory 35-ft channel has been maintained in Southwest Pass since 1923. September, 1959 conditions in the lower jettied sector of Southwest Pass and the sea bar channel are shown in Fig. 7.

1956-57 DATA COLLECTION PROGRAM

A pilot model, constructed and tested in 1948 by the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, showed that the salt-water wedge and associated saline conditions in the passes could be duplicated and that a comprehensive model study was feasible. In fiscal year 1955 the Congress allocated $175,000 to initiate planning studies to provide the 40-ft deep channel via Southwest Pass, authorized by the R & H Act approved March 2, 1945. The Division Engineer, U. S. Army Engineer Division, Lower Mississippi Valley, Vicksburg, Miss., directed that the plan of improvement be based on prototype data, engineering and navigation studies and model investigations. A comprehensive data collection program was undertaken in Southwest Pass and Sea Bar by the New Orleans District to provide basic data for

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[6] "Notes on the Mouth of the Mississippi River," by E. J. Dent, 1921.

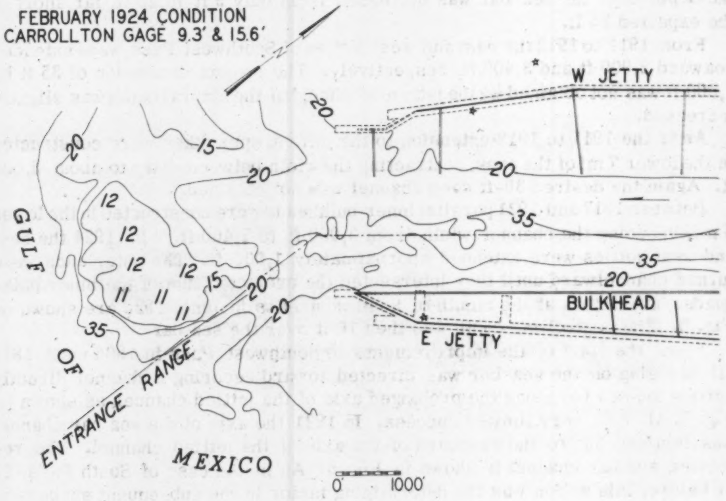
FEBRUARY 1924 CONDITION
CARROLLTON GAGE 9.3' & 15.6'
N
W JETTY
20
15
20
20
12
12
12
12
11
11
11
15
20
20
35
35
35
20
BULKHEAD
E JETTY
GULF OF MEXICO
ENTRANCE RANGE
0
1000'

FIG. 6.—FEBRUARY 1924 CONDITIONS, SOUTHWEST PASS AND SEA BAR CHANNEL

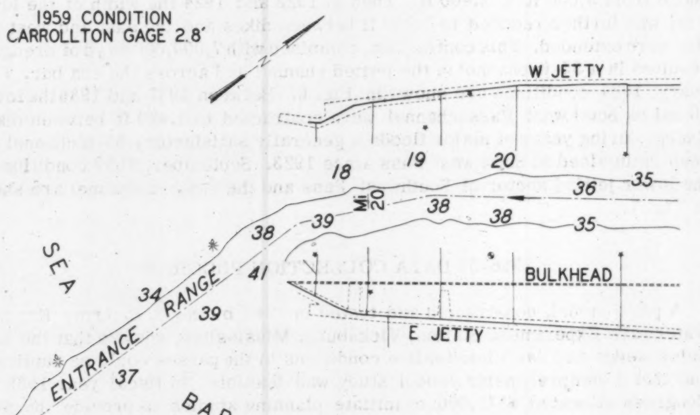
1959 CONDITION
CARROLLTON GAGE 2.8'
N
W JETTY
19
20
20
18
20 MI
35
36
38
38
35
39
38
41
BULKHEAD
34
39
E JETTY
SEA
ENTRANCE RANGE
BAR
37
36
0
400'

FIG. 7.—SEPTEMBER 1959 CONDITIONS, SOUTHWEST PASS AND SEA BAR CHANNEL

the prototype studies and data and information with which to plan, construct and verify a model of the lower 12 mi of Southwest Pass channel and Sea Bar channel. The data collection program was planned to obtain information on major factors influencing shoaling in tidal waters; namely the river's flow, including action of the salt-water wedge, channel alignment, and the Gulf tides. The field observations were made over a one year period to provide measurements during the complete hydrological cycle. Detailed data were obtained on the following: (1) flow in Southwest Pass (direction and magnitude); (2) salinity in pass channel and adjacent Gulf area; (3) sediment samples at various locations in the pass channel and adjacent Gulf area; (4) hydrographic surveys of channel and Gulf area; (5) Gulf flow currents; (6) tides; (7) winds; (8) waves; (9) past dredging requirements. (Data collected were compiled and published in two volumes by the New Orleans District in 1959.) The sectors of Southwest Pass and the Gulf covered in the data collection program are shown by Fig. 8. Fig. 9 depicts a marine stationary tower used in the program. Roberts radio current meter used in the program is shown in Fig. 10, and a recorder for a Roberts Meter in Fig. 11. Fig. 12 shows a small clamshell used to obtain bed material samples.

GULF TIDES

The Gulf tides are diurnal and have a range of 2 ft during spring tides. However, extreme tides caused by hurricane conditions have reached a stage of approximately 6 ft. At New Orleans, La. the average variation in low water river stage due to tides is approximately 10 in. to 12 in. During low water a slight tidal effect can be observed 35 mi above Baton Rouge, La.

FLOW CONDITIONS IN SOUTHWEST PASS

The fresh water discharge of the pass is controlled in part by Gulf tidal action. At low river stages the flow out of Southwest Pass may vary as much as 300%. Measurements showed a flow variation of 42,000 cfs to 128,000 cfs during one tidal cycle. At high river stages the tidal effect is minimized, and it may be ignored at flood stages. Normally, Southwest Pass carries 38% of the total river flow at the Head of Passes. A maximum flow of 354,000 cfs has been measured during flood conditions. An outflow of 24,900 cfs has been measured during low low-stage, slack water conditions. At this low stage the upstream flow of salt-water under the fresh-water layer was measured and found to be 25,900 cfs.[7] This upstream flow of Gulf water is hereinafter referred to as the "salt-water wedge."

SALT-WATER WEDGE PHENOMENON

Recorded measurements show that at extreme low stages the salt-water wedge may penetrate into the Mississippi River as much as 135 mi above the mouth of Southwest Pass, a fact which, at times, affected the New Orleans

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[7] "Investigations and Data Collection Program for Model Study of Southwest Pass, Mississippi River," New Orleans Dist., April, 1959.

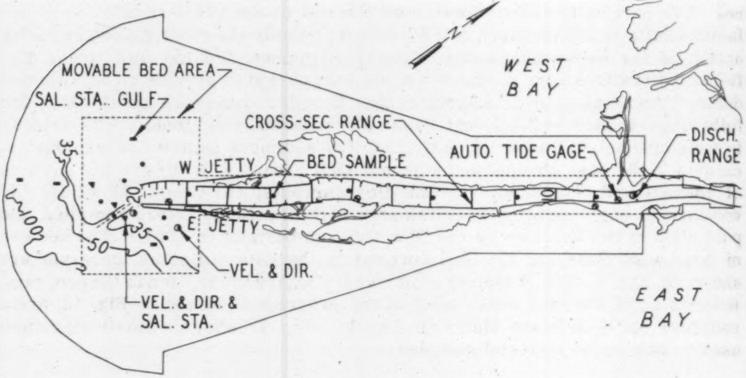
N
WEST BAY
MOVABLE BED AREA
SAL. STA.. GULF
CROSS-SEC. RANGE
AUTO. TIDE GAGE
DISCH. RANGE
W. JETTY
BED SAMPLE
E. JETTY
VEL. & DIR.
VEL. & DIR. & SAL. STA.
EAST BAY
35
100
50
20
35
10

FIG. 8.—AREA COVERED IN DATA COLLECTION PROGRAM

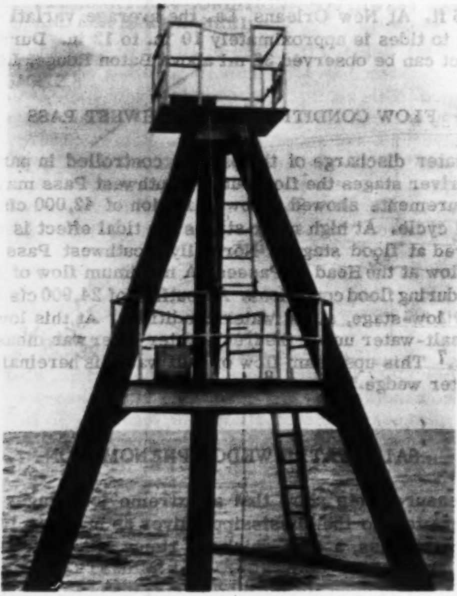

FIG. 9.—MARINE TOWER USED IN DATA COLLECTION PROGRAM

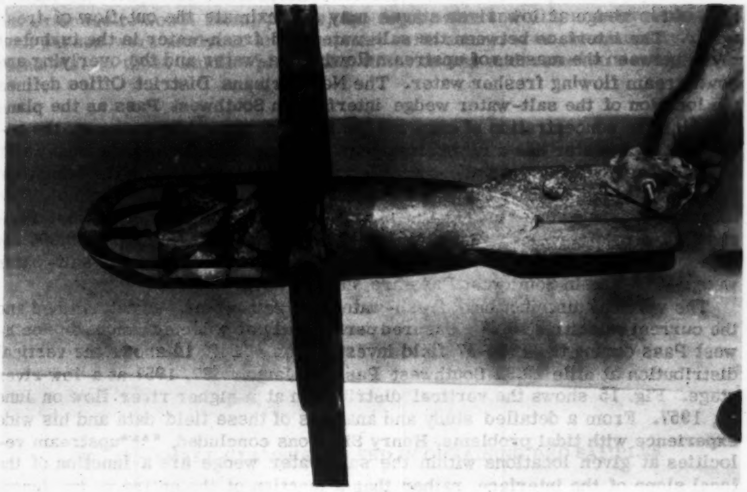

FIG. 10.—ROBERTS CURRENT METER USED IN DATA COLLECTION PROGRAM

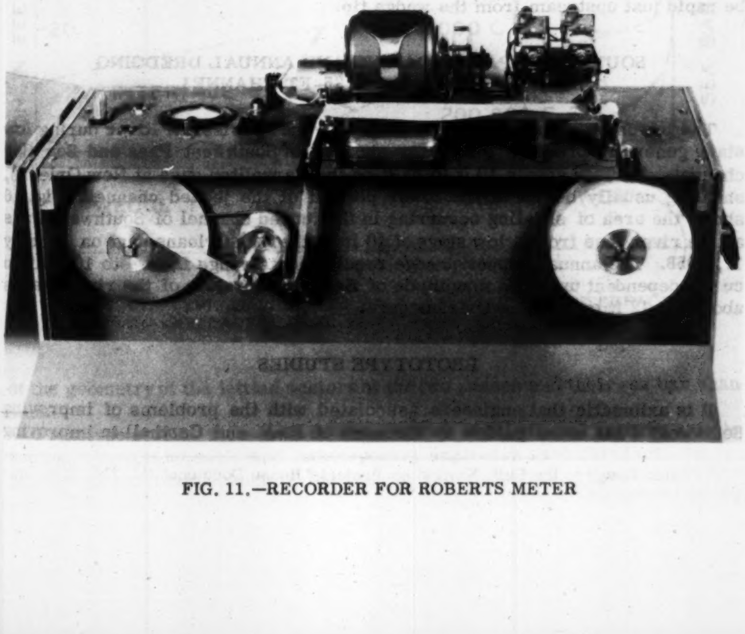

FIG. 11.—RECORDER FOR ROBERTS METER

water supply.[8] As previously noted, in Southwest Pass the upstream flow of salt-water wedge at low river stages may approximate the out-flow of fresh water. The interface between the salt-water and fresh-water is the turbulent layer between the masses of upstream flowing sea-water and the overlying and downstream flowing fresher water. The New Orleans District Office defines the location of the salt-water wedge interface in Southwest Pass as the plane at which the concentration of chloride ion is 5,000 ppm.[7] By contrast, the undiluted fresh-water has a concentration of 30 ppm or less, and the sea-water near the mouth of the pass about 20,000 ppm. In Southwest Pass the transition from low to high salinity takes place in only a few feet in the vertical. The locations of the wedge tip and the slopes of the interface in Southwest Pass channel were determined for fresh-water discharges ranging from 30,000 cfs to about 300,000 cfs in Southwest Pass.[7] Typical profiles of the salt-water wedge interface in Southwest Pass are shown in Fig. 13.

The vertical distribution of fresh-water and salt-water was determined and the current velocities were measured periodically at selected ranges in Southwest Pass during the 1956-57 field investigation.[7] Fig. 14 shows the vertical distribution at mile 19.92 Southwest Pass on January 27, 1957 at a low river stage. Fig. 15 shows the vertical distribution at a higher river flow on June 12, 1957. From a detailed study and analysis of these field data and his wide experience with tidal problems, Henry Simmons concluded, "***upstream velocities at given locations within the salt-water wedge are a function of the local slope of the interface, rather than a function of the entire wedge length as had previously been supposed, so that maximum upstream velocities in the jettied and sea bar entrance channel occur during the time of maximum fresh-water discharge and minimum wedge length." This high water action of the salt-water wedge under the fresh-water outflow provides an effective barrier to bed-load movement in the lower two miles of the jettied channel. It is here that the heavy annual shoaling occurs during high river stages - postulated to be rapid just upstream from the wedge tip.

SOUTHWEST PASS SHOALING AND ANNUAL DREDGING REQUIREMENTS FOR 35-FT CHANNEL

The intrusion of the salt-water wedge into the Mississippi River during low stage generally prevents low water shoaling in Southwest Pass and Sea Bar channel. At river stages 10 ft to 12 ft on the Carrollton gage at New Orleans, shoaling usually begins in the lower portion of the jettied channel. Fig. 16 shows the area of shoaling occurring in the jettied channel of Southwest Pass as the river rose from a low stage of 10 ft on the New Orleans gage on January 17, 1958. The annual hopper dredge requirements range from 4 to 10 million cu yd, dependent upon the magnitude of flow and duration of the river stages above 10-12 ft on the New Orleans gage.

PROTOTYPE STUDIES

It is axiomatic that engineers associated with the problems of improving Southwest Pass were guided by the work of Eads and Corthell in improving

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[8] "Baton Rouge to the Gulf, Navigation Project," House Document No. 215, 76th Congress, 1st Session.

South Pass.[5] Although field investigations and the data collection program were not extended to South Pass, per se, the depth, with and alignment relations of both passes were analyzed and compared with attention to the sinuosity-depth relations. In these studies particular attention was addressed to a comparison of the geometry of the jettied sectors of the two passes and their sea bar channels.

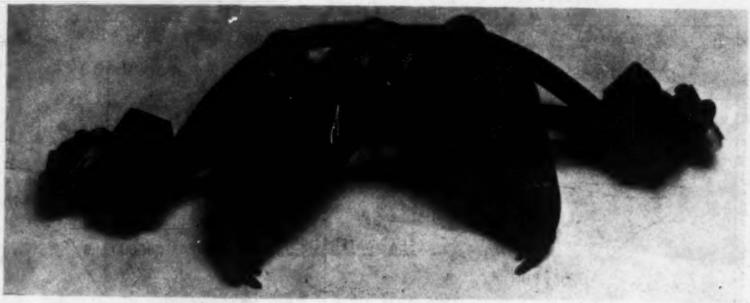

FIG. 12.—SMALL CLAMSHELL USED IN OBTAINING BED SAMPLES

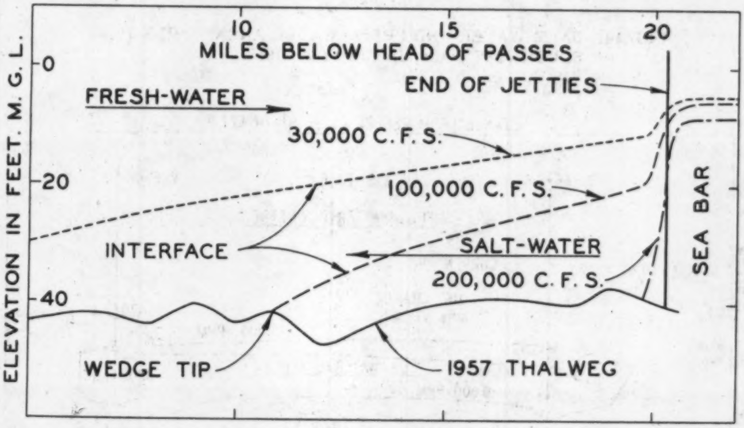

FIG. 13.—TYPICAL PROFILES, SALT-WATER WEDGE INTERFACE, SOUTHWEST PASS

Prototype studies included a re-examination of all official reports, articles and manuscripts by early and contemporary engineers associated with the improvement and maintenance of the passes, record survey maps, construction records and the dredging history as concerns Southwest and South Passes. Prior investigations and concepts of the causes and methods devised for lessening the annual shoaling in Southwest Pass and Sea Bar channel were examined. Schultz' definitive sediment observations[9] and Dents' observations on

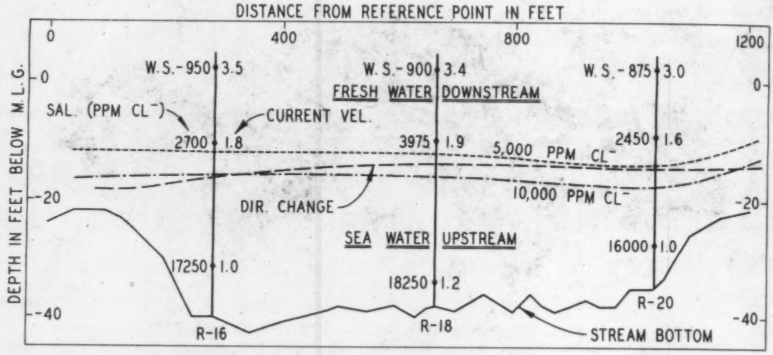

FIG. 14.—SALT-WATER AND FRESH-WATER AREAS—MILE 19.92 SOUTHWEST PASS, JANUARY 27, 1957

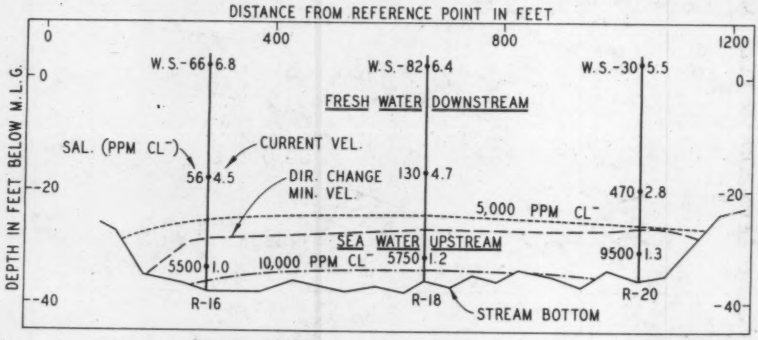

FIG. 15.—SALT-WATER AND FRESH-WATER AREAS—MILE 19.92 SOUTHWEST PASS, JUNE 12, 1957

the salt-water wedge[6] were correlated with the field investigations and data collection program.

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[9] "Sediment Observations, Passes of the Mississippi River," by Schultz, 1921.

GENERAL CONCLUSIONS FROM PROTOTYPE STUDIES AND FIELD INVESTIGATIONS

The following general conclusions were reached from the prototype studies and collating field observations:

1. Maintenance of a 40-ft channel through the existing Southwest Pass channel by dredging only would be difficult and expensive. It is estimated present hopper dredge requirements to maintain the 35-ft depth channel would be increased by 2 to 3 hopper dredge months. During major floods additional hopper dredge capacity would be required to maintain a dependable 40-ft channel.

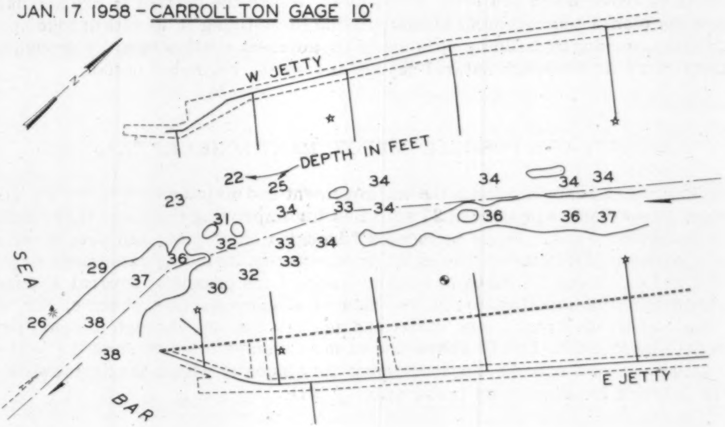

FIG. 16.—17 JANUARY 1958 CONDITIONS AT SOUTHWEST PASS

2. Premised on the excellent results of the Eads-Corthell improvement work at South Pass and an analysis of the existing channel alignment and channel areas, widths and depths of South and Southwest Passes, a more sinuous alignment will be required through the lower half of Southwest Pass, particularly below Mile 17 BHP, to secure a self-maintaining 40-ft ship channel.

3. During flood stages the Mississippi River carries a heavy bed load and a substantial sediment load. The existing high water flow regimen is favorable to deposition of the bed load and suspended sediment load materials in the lower sector and at the mouth of Southwest Pass.

4. During low water when the bed and suspended sediment load of the Mississippi River is greatly reduced, the upstream intrustion of the salt-water wedge seemingly acts as a shield against shoaling in the pass channel.

5. Under existing high water conditions the salt-water wedge acts as a barrier to bed-load movement beyond the wedge tip, and heavy shoaling may follow the wedge tip in its play upstream and downstream in the lower jettied channel and at the mouth of Southwest Pass, dependent upon fresh-water flows and tidal influences.

6. To be effective channel improvements must modify the high-water, fresh-water flow—salt-water wedge relation to alleviate the heavy high-water shoaling that now occurs in Southwest Pass. To provide a self-maintained channel through Southwest Pass, the channel improvement design must give more muscle to the high-water, fresh-water flow.

7. The law batture banks of the passes should be kept strong enough to prevent crevassing. They are subject to overflow to a depth of about one foot during floods. There is a slight annual subsidence of the batture banks, but river deposits during overbank flows are generally sufficient to maintain the batture banks to above mean sea level elevations. Where the batture is low and narrow the plan of improvement should provide for building it up with dredge spoil or strengthening by building up lands in the adjacent shallow bays by deposition from river flow through natural bayous and small, regulated outlets.

TWENTY-ONE POSSIBLE IMPROVEMENT SCHEMES EXAMINED

Engineers associated with the improvement and maintenance of the Mississippi River Passes presented 21 schemes for improving the lower three miles of Southwest Pass. These included a "dredging only" plan, and several plans proposing modifications to the existing structures, including extensions to jetties and extending the dikes to further contract the channel. Several schemes proposed some modification of the channel alignment. One scheme, Fig. 18, proposed to construct a new outlet and sea bar channel through the east jetty near mile 19 BHP. Fig. 17 shows one of the schemes for extending the jetties. Fig. 19 shows a scheme for increasing the sinuosity within the framework of the existing improvements below mile 17 BHP.

MODEL INVESTIGATIONS

The unique tidal model of lower Southwest Pass and Sea Bar channel was designed and operated at the U. S. Army Engineer Waterways Experiment Station by Messrs. H. B. Simmons and H. J. Rhodes, using the field data collected during 1957-58. The model tests provided excellent qualitative information on the shoaling problem. The fixed-bed model, scale 1: 500 horizontally and 1: 100 vertically, accurately reproduced the salt-water wedge phenomenon and duplicated the high-water shoaling pattern of the existing channel with colored plastic bed-load materials introduced at the upper end.

COMPLETE TESTS ON TWO PROPOSED PLANS

In model tests of the 21 improvement schemes, Plans 14 and 17A offered the best possibilities for lessening the heavy annual high water shoaling. Plan 14 is shown on Fig. 19. Plan 17A is shown on Fig. 18. These two plans were tested for conditions of river flow and Gulf tides representative of the full range

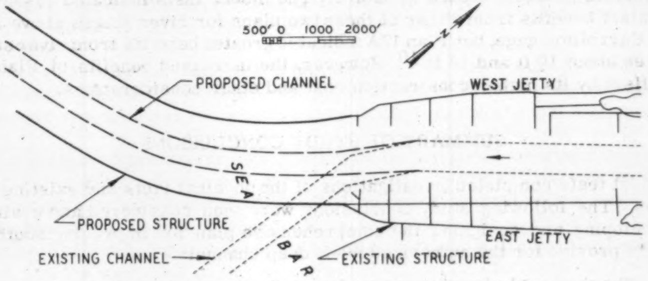
500' 0 1000 2000'
N
PROPOSED CHANNEL
WEST JETTY
SEA BAR
PROPOSED STRUCTURE
EAST JETTY
EXISTING CHANNEL
EXISTING STRUCTURE

FIG. 17.—SCHEME FOR EXTENDING JETTIES

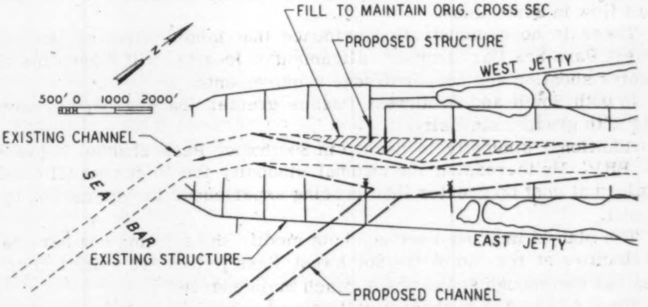
FILL TO MAINTAIN ORIG. CROSS SEC.
PROPOSED STRUCTURE
N
WEST JETTY
500' 0 1000 2000'
EXISTING CHANNEL
SEA BAR
EAST JETTY
EXISTING STRUCTURE
PROPOSED CHANNEL

FIG. 18.—SCHEME FOR NEW SEA BAR CHANNEL IN VICINITY MILE 19, SOUTHWEST PASS

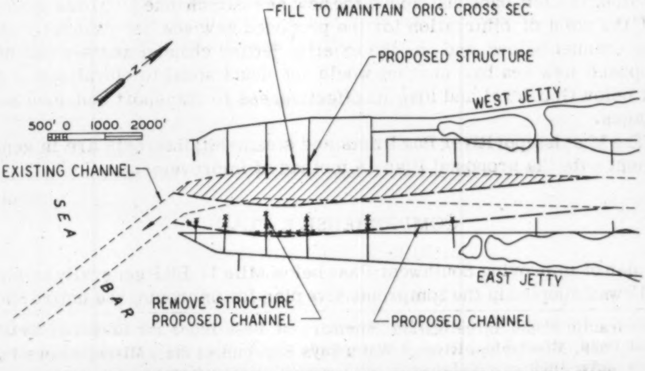
FILL TO MAINTAIN ORIG. CROSS SEC.
N
PROPOSED STRUCTURE
WEST JETTY
500' 0 1000 2000'
EXISTING CHANNEL
SEA BAR
EAST JETTY
REMOVE STRUCTURE
PROPOSED CHANNEL
PROPOSED CHANNEL

FIG. 19.—SCHEME FOR ADDITIONAL SINUOSITY BELOW MILE 17, SOUTHWEST PASS

that serious shoaling occurs in nature. The model tests indicated essentially equivalent benefits from either of these two plans for river stages above about 17 ft, Carrollton gage, but Plan 17A indicated greater benefits from river stages between about 10 ft and 16 ft.[10] However, the increased benefits of Plan 17A are offset by its higher construction cost and other considerations.

SUMMARY OF STUDY CONCLUSIONS

Model tests completed investigations of the physical facts and existing conditions. The following study conclusions were then considered and evaluated in developing and designing the comprehensive plan for improving Southwest Pass to provide for the authorized 40-ft deep channel:

1. The abnormal elevation of shoaling in Southwest Pass during high stages correlates closely with the transient locations of the salt-water wedge tip.
2. High water shoaling rarely occurs in South Pass channel, and annual dredging, if any, is nominal—seemingly the high fresh-water flow dominates the Gulf flow in South Pass.
3. There is no demonstration evidence that modification of the present Southwest Pass Sea Bar channel's alignment or location will lessen the annual high water shoaling and hopper dredge requirements.
4. In both South and Southwest Passes greater thalweg depths correlate closely with greater sinuosity.
5. Additional sinuosity is needed in Southwest Pass channel below about mile 9 BHP. By increasing the channel sinuosity the 40-ft channel should be maintained at cost no greater than is being experienced in maintaining the 35-ft channel.
6. The plan of improvement need not modify the existing sea bar channel. The velocities at the end of the Southwest Pass jetties are sufficient to keep the sea bar far enough seaward for reach maintenance.
7. Plan 14 (Fig. 19) is theoretically sound and is supported by both the excellent results obtained by the channel improvement at South Pass and by the model tests.
8. Plan 17A (Fig. 18) is supported by the model test and is theoretically sound, but it is quixotic with respect to: (a) the maintenance of the sharp curvature along the left bank just above the new sea bar channel; (b) the maintenance of the point of bifurcation for the proposed new sea bar channel with the existing channel below; and (c) the existing jettied channel and sea bar below the proposed new sea bar channel would not doubt shoal to elevations of 10 ft to 15 ft below Gulf level and lose its effectiveness to transport bed-load during high stages.
9. The Mississippi River Bar Pilots and steamship interests are in general agreement with the proposed Plan 14 method of improvement.

COMPREHENSIVE PLAN

The plan of improving Southwest Pass below Mile 17 BHP generally as shown in Fig. 19 was adopted in the comprehensive plan for improving the entire Southwest Pass channel. The structures, dredging and other construction required to provide the 40-ft channel via Southwest Pass are:

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[10] "Hydraulic Model Investigation, Summary of Best Plans for Reducting Shoaling, Southwest Pass, Mississippi River," Waterways Experiment Sta., Miscellaneous Paper No. 2-349, July, 1959.

Dredging.—

1. Between mile 1.1 AHP in the Mississippi River and mile 17.5 BHP, the 40-ft channel (with 2 ft overdepth) will be obtained by dredging where existing dimensions are deficient and where a modification in the channel alignment is required. An estimated 13,000,000 cu yd of material will be removed by cutterhead dredge. Here the dredge spoil will be deposited overbank on either side of the channel to raise and strengthen areas of low and narrow batture.

2. Between mile 17.5 and mile 20.2 BHP the 40-ft channel will be excavated by cutterhead and hopper dredges. It is estimated that 4,600,000 cu yd of dredging will be required. Where practicable, cutterhead dredge spoil may be deposited on the west side of the pass channel within the dike system to retain, insofar as possible, the existing cross-sectional area of the channel. Otherwise, the spoil will be placed on the east side of the channel.

3. The 40-ft by 600-ft sea bar channel running from Mile 20.2 BHP to the 40-ft contour in the Gulf of Mexico will be obtained by hopper dredging. Some 2,300,000 cu yd of dredging will be required.

Dikes.—The existing lateral pile dike system will be modified by lengthening some of the existing dikes, by construction of new dikes and removal of some dikes. A new dike system may be required along the right bank between mile 2 and mile 3 BHP, to contact and realign the channel in this reach. Between mile 9.0 and mile 20.2 BHP the existing dikes along the west bank will be lengthened and new dikes added. The portion of the dike work between mile 9.0 and mile 12.3 is designed to strengthen and protect the narrow batture banks from crevassing and wave wash erosion. Between mile 12.3 and mile 20.2 BHP the dike work is designed to promote the development and stabilization of the channel along the more sinuous alignment, particularly in the reach below mile 17.5 BHP. Removal of portions of the dikes along the east bank between mile 18 and mile 20 BHP is necessary to effect this realignment. A total length of approximately 13,000 ft of new dike extension are required. Approximately 3,000 ft of existing dikes will be removed.

Bulkheads.—A longitudinal bulkhead of the Wakefield type will be constructed to protect the badly eroded west bank between mile 15.3 and mile 16.1 BHP.

Relocations.—The proposed realignment of the channel in the lower two miles of Southwest Pass will require the relocation of navigation aids, and minor modifications of the U. S. Coast Guard facilities.

ESTIMATE OF COST

The total estimated Federal cost of the project is $8,300,000, not including associated Federal cost of $140,000. Non-Federal first cost for relocations is $18,000.

CONSTRUCTION SCHEDULE

It is estimated that construction will require approximately 3-1/2 years. The dredging work at the entrance to Southwest Pass will be conducted in two phases. In the first phase the 35-ft channel will be shifted to the new easterly alignment by dredging and modification of the channel contraction dikes. Deepening of the channel to 40 ft will be accomplished in the second phase. The longitudinal bulkhead between miles 15 and 16 BHP will be constructed in the early part of the construction period. Construction of other training works and structures in the pass will be initiated in the latter part of the construction period.

BENEFITS

The trend of modern shipping is toward larger, deeper draft vessels. Channel depths in excess of 40 ft exist from a short distance above Head of Passes to New Orleans. The 40-ft channel will permit ships to load to deeper drafts, thus utilizing all cargo space, and will eliminate the hazards of deep draft vessels navigating in Southwest Pass due to fluctuation in depths in the pass. The latest approved benefit-to-cost ratio for the project, Mississippi River, Baton Rouge to the Gulf of Mexico, La., is 16.7 to 1 (May 1958). This ratio as currently estimated is 18.5 to 1, due to increased waterway tonnage. Benefits due to the incremental increase in channel depth from -35 to -40 ft between New Orleans and the Gulf of Mexico were evaluated separately.

ACKNOWLEDGMENTS

The Southwest Pass improvements are in accordance with the plan recommended by Col. Cookson, District Engineer, New Orleans, and Gen. Carter, Division Engineer, Vicksburg, and approved by Gen. Itschner, Chief of Engineers, Corps of Engineers.

The author is indebted to a number of people in the New Orleans District, the Waterways Experiment Station and the Lower Mississippi Valley Division for advice, suggestions and in preparing the paper. He particularly wishes to acknowledge the advice and suggestions of Messrs. Rhodes, Simmons, Gentilich, Kennedy, Toffaleti, McCleave, Miss Jones and Mrs. McQuiller.

APPENDIX.—ADDITIONAL REFERENCES

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2. "Problems at the Mouth of the Mississippi River," by Townsend, Military Engineer, March-April, 1926.
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5. "Comments on Shoaling Problems, Southwest Pass, Mississippi River," by Henry B. Simmons, Waterways Experiment Sta., Miscellaneous Paper No. 2-155, February, 1956.
6. "Fresh-water—Salt-water Density Currents, Major Cause of Siltation," by Schultz and Simmons, Permanent Internatl. Assn. of Navigation Congresses, 1957.

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TRANSACTIONS

Paper No. 3170

PLAN FOR CLOSURE OF OLD RIVER

By George M. Cookson,[1] M. ASCE

SYNOPSIS

The closure of Old River is the crucial feature of the project for flood control and improvement of the lower Mississippi River. This operation involves an initial rock closure, followed by a final hydraulic fill. Geological features and hydraulic characteristics require this procedure in order to provide maximum assurance of positive closure within economical limitations.

INTRODUCTION

The structures for the control of flows from the Mississippi River in the vicinity of Old River are planned primarily to prevent the Mississippi River from abandoning its present channel below Old River and adopting the course of the Atchafalaya River. The primary intention is to prevent the change of course of the Mississippi River by the interposition of safe and stable structures and by the control of flows to suitable volumes; and, with these same flows, to enlarge and maintain at no more than a safe value, the flood-carrying capacity of the Atchafalaya River.

The authorized project for Old River control provides for construction of a controlled diversion system north of Old River, completion of the right bank Mississippi River levee system in the vicinity of Old River to full grade, and a closure dam in Old River. The new diversion system consists of a low-sill structure in the levee line with appropriate inflow and outflow channels to control flows below bankfull stage, and an overbank control structure to be operated for the diversion of excess flood flows from the Mississippi River. A

Note.—Published essentially as printed here, in September, 1960, in the Journal of the Waterways and Harbors Division, as Proceedings Paper 2602. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Col. and Dist. Engr., U. S. Army Engr. Dist., New Orleans, New Orleans, La.

navigation lock and connecting channels are under construction on the south bank of Old River, adjacent to the closure, to provide continuation of existing commerce.

LOCATION AND DESCRIPTION

The junction of Old River with the Mississippi is approximately 302 miles above Head of Passes. It is at the latitude of the Louisiana-Mississippi State line, (Fig. 1), about 60 miles northwest of Baton Rouge, Louisiana.

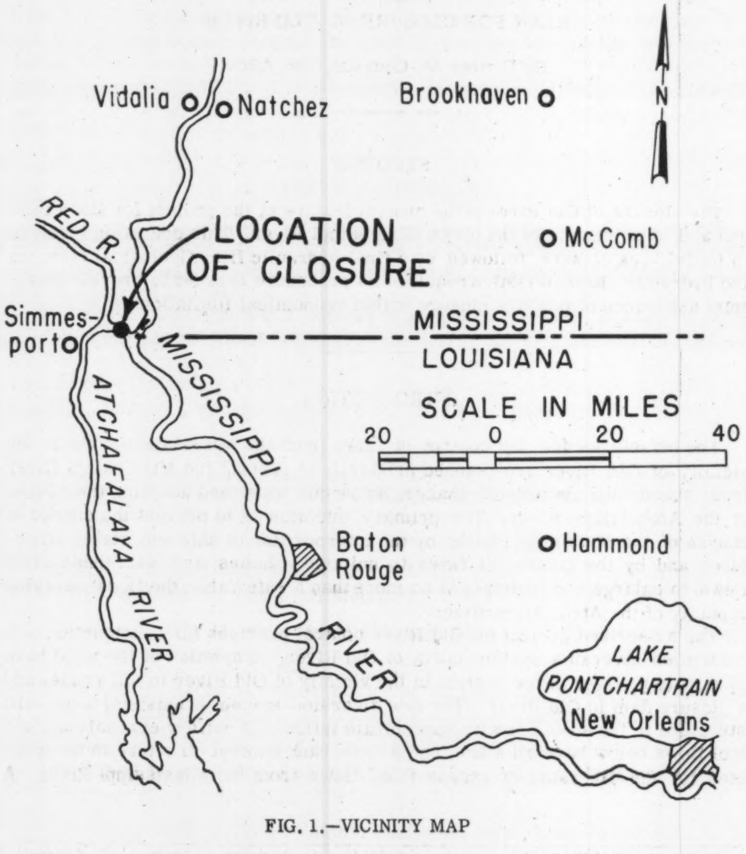

Vidalia
Natchez
Brookhaven
N
RED R.
LOCATION OF CLOSURE
McComb
Simmesport
MISSISSIPPI
LOUISIANA
ATCHAFALAYA RIVER
MISSISSIPPI RIVER
SCALE IN MILES
20 0 20 40
Baton Rouge
Hammond
LAKE PONTCHARTRAIN
New Orleans

FIG. 1.—VICINITY MAP

The closure dam in Old River is located approximately 2,100 ft downstream from the site of the former Texas and Pacific Railroad Bridge (Fig. 2). The location is about 1 1/2 miles from the junction of the Mississippi River and Old River, about 5 1/2 miles from the junction of the Red, Atchafalaya, and Old Rivers, and about 6 1/2 miles southwest of the low-sill and overbank control structures (Fig. 3). The location of the closure is adjacent to the site of

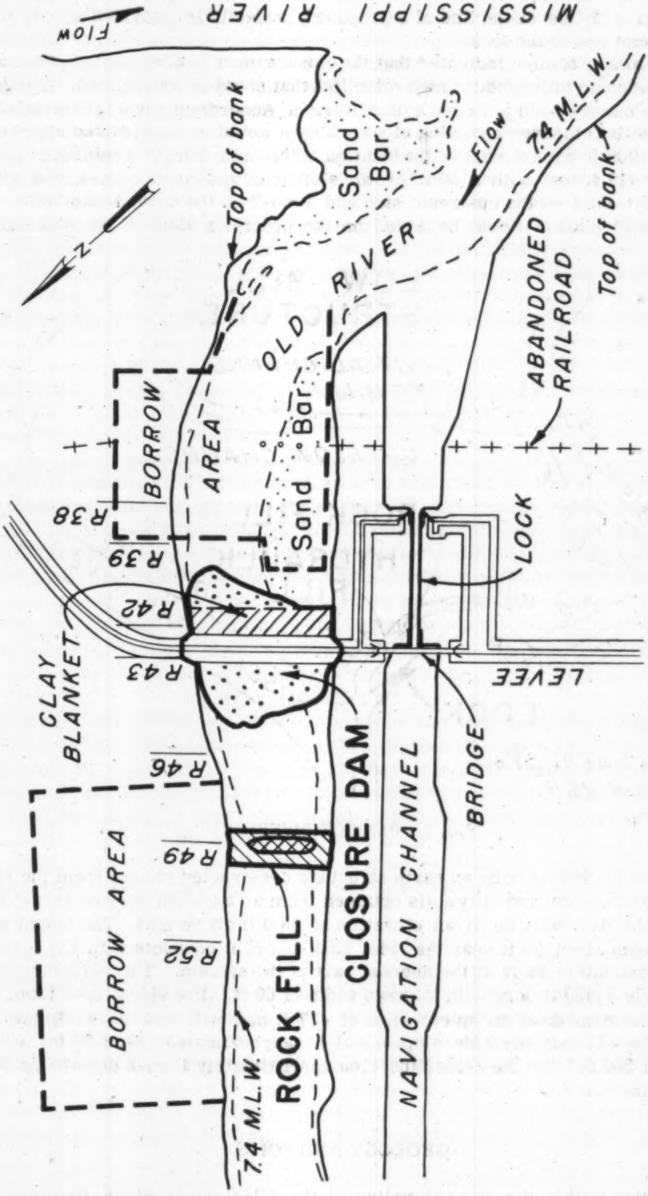

MISSISSIPPI RIVER
FLOW
Top of bank
Sand Bar
Flow
7.4 M.L.W.
N
OLD RIVER
ABANDONED RAILROAD
Top of bank
BORROW AREA
Sand Bar
R38
R39
LOCK
R42
CLAY BLANKET
R43
LEVEE
CLOSURE DAM
BRIDGE
R46
NAVIGATION CHANNEL
R49
BORROW AREA
R52
ROCK FILL
7.4 M.L.W.

FIG. 2.—LOCATION OF CLOSURE

the lock now under construction (1960). The final, or main dam will be in alignment with the centerline of a proposed bridge to be provided across the downstream end of the lock.

Preliminary studies indicated that the closure must be constructed of a material that could withstand the high velocities that could be anticipated. Locally available material did not meet this criterion. Accordingly, it was determined that an initial closure consisting of a rock dam would be constructed approximately 1,900 ft downstream of the location of the main dam, to a minimum elevation of +17 ft msl with a 30 ft crown sloping downstream 1-on-3, and with slopes of 1-on-1 on the upstream side and 1-on-2 on the downstream side. A graded seal blanket would be added on the upstream slope of the rock dam.

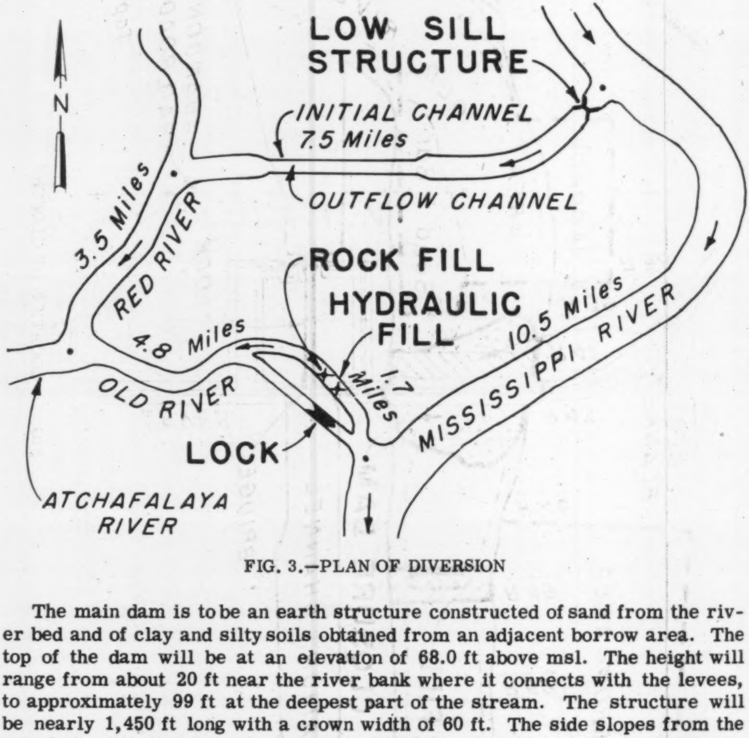

N
LOW SILL STRUCTURE
INITIAL CHANNEL 7.5 Miles
OUTFLOW CHANNEL
3.5 Miles
RED RIVER
ROCK FILL
HYDRAULIC FILL
10.5 Miles
MISSISSIPPI RIVER
4.8 Miles
1.7 Miles
OLD RIVER
LOCK
ATCHAFALAYA RIVER

FIG. 3.—PLAN OF DIVERSION

The main dam is to be an earth structure constructed of sand from the river bed and of clay and silty soils obtained from an adjacent borrow area. The top of the dam will be at an elevation of 68.0 ft above msl. The height will range from about 20 ft near the river bank where it connects with the levees, to approximately 99 ft at the deepest part of the stream. The structure will be nearly 1,450 ft long with a crown width of 60 ft. The side slopes from the top of the dam down to an elevation of + 37 ft msl will be 1-on-4. Below the elevation + 37 msl the side slopes will be approximately 1-on-20 for a distance of 500 ft from the centerline, then approximately 1-on-6 down to the bottom of the river.

GEOLOGY AND SOILS

Located within the alluvial valley of the Mississippi River, the geologic history of Old River is an integral part of the history of the Mississippi. As a result of cyclic glaciation and variations in sea level during the Pleistocene epoch, the Mississippi River in the vicinity of Old River and elsewhere in the Alluvial Valley became deeply entrenched in older sedimentary beds. Later, the entrenched valley was partially refilled and buried with sediments that were washed into the entrenchment by numerous tributary streams and graded by the river. During the early stage of valley refilling, the Mississippi transported and deposited relatively coarse materials as the aggrading valley surface developed. As the deposit in the entrenched valley thickened, the stream gradient was reduced, and the alluvium that was carried by the river and deposited became finer. The river changed from a braided to a meandering stream with systematic channel migration that resulted in the formation of conspicuous meander belts and adjacent low lands or back-swamps.

The material filling the alluvial valley has been divided into a substratum consisting of clean sands with some gravel, and a top stratum consisting of clays, silts, and silty sands. Based on the manner in which the sediments were deposited and on the characteristic gradations, the top stratum has been subdivided into a point bar deposit consisting predominately of silty sands, a channel filling deposit consisting of lean clays and silty sands, a back-swamp deposit composed mostly of fat clays, and a natural levee deposit composed mostly of sandy silts and silt.

Old River is located across the western half of the meander belt of the present course of the Mississippi, an area composed of natural levee, channel filling, and point bar deposits underlain by a substratum of clean sand. Prior to 1831 a large meander loop of the Mississippi River occupied the area, and the lower or southern limb of the loop was located along the present course of Old River. At that time the Red and Atchafalaya Rivers were directly connected with the Mississippi. The Red River joined the Mississippi at the northwest section of the meander loop, and two miles downstream from this junction the Atchafalaya River formed and flowed away from the Mississippi. In 1831, Capt. Henry M. Shreve cut a channel through the narrow neck of land at the eastern end of the loop which expedited a natural process and caused most of the flow in the Mississippi River to by-pass the loop and follow the shorter cut-off route. Some flow continued through both the upper and lower limbs of the loop, but reduction in flow through these channels caused sediments to be deposited and partially fill them. Finally, aided by some dredging at the mouth of the lower limb, the upper limb silted-up and the lower limb became the only connection between the Mississippi, Red, and Atchafalaya Rivers. The present channel of Old River is, therefore, a channel, within a large channel of the Mississippi, which has been partially refilled with sediments.

FIELD EXPLORATIONS

General-type borings were made in Old River and along its banks, in the area where the closure is located. These borings extended to elevations of -60 ft to -120 ft msl. Driving resistances on a split spoon sampling device were obtained on a representative number of borings penetrating the sand strata. Two 5-in. diam undisturbed borings were made in the area of the final closure to elevations -68 and -56, respectively, and undisturbed samples were obtained from the clay strata in the foundation. General type borings were made in the sand bar at the entrance to Old River, opposite Carr Point revetment, and in the sand bar on the east side of the Mississippi River opposite the entrance to Old River. These borings extended to elevations -65 ft to -70 ft msl. General type borings extending to elevations +24 to +30 were made in the borrow area on the north side of Old River adjacent to the final closure site.

SOIL CONDITIONS

The banks of Old River in the vicinity of the closure site consist generally of a surface layer of sandy silt and silty sand 5 ft to 20 ft thick overlying a layer composed of lean and fat clays ranging from 20 ft to 50 ft thick, which is underlain by fine sand. There is a deposit, predominately of clay, varying from 20 ft to 40 ft thick lining the bed of the river extending from bank to bank in the vicinity of hydrographic range 49, (Fig. 2). The clay deposit extends downstream about 900 ft and approximately the same distance upstream. Progressing further upstream the clay in the river bed becomes thinner and, at a point 2,700 ft upstream from range 49 and continuing towards the Mississippi, there is no clay, as disclosed by the borings taken in this reach. It was noted, as disclosed by borings, that the clay at the bottom of the river, as shown by the sections of ranges 39 and 49, is not continuous between these ranges. A fine grained sand bar centered on range 39 extends for a distance of about 3,000 ft. Driving resistances, measured with a standard split spoon sampler, indicate that the sand in Old River varies from loose, where it is exposed, to dense and very dense at elevation about -60 ft msl. The bar at the entrance to Old River opposite Carr Point Revetment and the bar on the east side of the Mississippi River opposite the entrance to Old River consist of fine sand.

The borrow area on the north side of Old River, adjacent to the closure area, is predominately fat clay to a depth of about 20 ft below ground surface. Beneath this surface clay stratum there is generally a stratum composed of sandy silt and silty sand which changes gradually with depth, to fine sand at elevations varying between -3 and +25. Figs. 4 and 5 show sections of the initial and final closures and the results of borings at these locations.

LABORATORY TESTS

Visual classification and water content determinations were performed on samples from all borings. Sieve analyses were made on typical sand samples and representative gradation curves of these findings were plotted. Permeability tests were run on composite sand samples from borings that were representative of the sand to be used in the main closure. Consolidation, unconfined compression, and shear tests were made on typical undisturbed clay samples from borings made in the area of the final closure. Unconfined compression tests were also conducted on small core samples of the clays obtained from general type borings.

HYDRAULICS

The direction and quantity of flow through Old River is influenced by the stage in the Mississippi River at the mouth of Old River, and the stage at the junction of the Red, Atchafalaya, and Old Rivers. Prior to 1916, flow in Old River was alternately to and from the Mississippi, but since that time the number of days of flow toward the Mississippi River has gradually decreased, so that since 1942, with the exception of 9 days in 1945, the flow has been from the Mississippi River to the Atchafalaya River. Prior to the final closure the overbank structure, the low-sill structure, and the connecting channels will be completed. Remaining to be completed (as of 1960) before Old River can be closed is a channel connecting the Mississippi River with the low-sill structure, and the levees along the Mississippi River to the ends of the closure dam.

DIVERSION OF FLOW

In the process of closing Old River, the low-sill structure will be opened. The flow diverted from the Mississippi River to the Red River through the low-sill structure will raise the stage at the lower end of Old River and, consequently, reduce the flow from the Mississippi into Old River. As material is placed in Old River to make a closure fill, or after a submerged partial closure fill has been constructed and the stage in the Mississippi River is falling, the discharge in Old River will decrease until final closure is obtained.

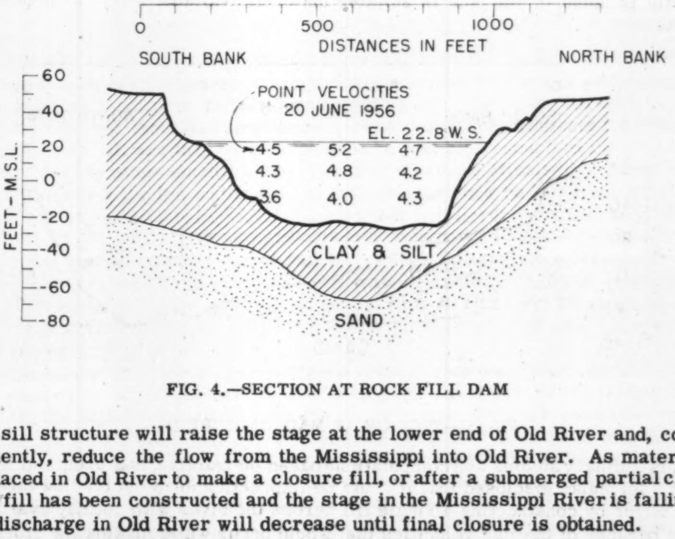


FIG. 4.—SECTION AT ROCK FILL DAM

The Red River is likely to rise rapidly during a flash flood occurring at any season of the year and reduce the flow through Old River. It does not follow as regular a seasonal discharge sequence as the Mississippi River. Although some flow from the Red River can be anticipated during the season when the Mississippi River is at a stage of about +45 (by chance this flow could be large enough to advantageously alter the stage at the mouth of Red River), the quantity that can be expected from the Red River with reasonable assurance will not exceed 15,000 to 20,000 cfs. This would raise the tailwater of the closure dam less than 1.0 ft. Therefore, the flow from Red River was not included in the study for either highwater or low water conditions of flow. The use of the lock to divert flow is not contemplated, but the culverts in the lock walls could be used for short durations in emergencies. Since the soils comprising the lower part of the lock channels are easy to erode, prolonged flows with relatively high velocity through the channel would result in serious scouring.

DESIGN OF CLOSURE

Initial consideration was given to a plan whereby a fill would be constructed of successive layers of sand, gravel, and rock, each to the limit of its stability under prevailing velocities. This would be done on the alignment of the levees and of the bridge over the lock. Involved, however, would be the difficulty of controlling the placement of materials to the extent necessary to prevent a crevasse from occurring, especially while the rock is being placed. If a crevasse should occur through the rock while the closure is being made, it would extend rapidly into the finer grained gravel and sand and make it impossible to prevent the scouring out of the greater part of the fill. No further consideration was given this plan, due to the inherent risk involved.

Study was made of a plan to make a single complete dredge fill on final alignment using pea gravel, clay, or coarse sand. This was discarded primarily because of the lack of suitable material available within an economic distance.

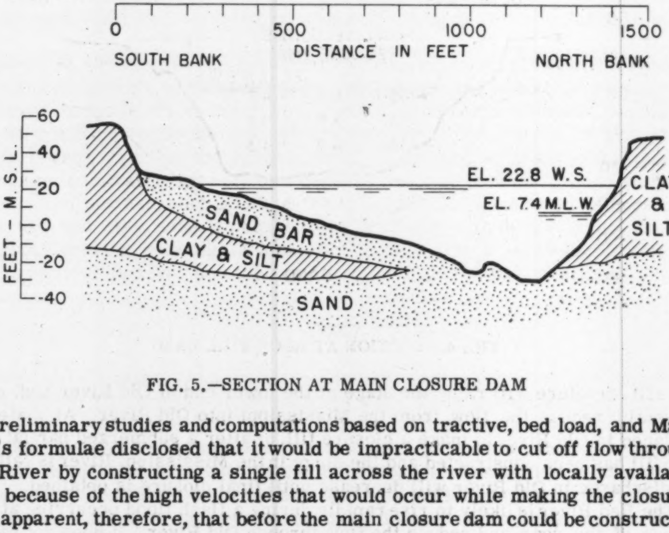


FIG. 5.—SECTION AT MAIN CLOSURE DAM

Preliminary studies and computations based on tractive, bed load, and Manning's formulae disclosed that it would be impracticable to cut off flow through Old River by constructing a single fill across the river with locally available sand because of the high velocities that would occur while making the closure. It is apparent, therefore, that before the main closure dam could be constructed the flow would have to be cut off by constructing an initial closure fill. The following methods were considered for making the initial closure:

1. Construct a clay fill during the low water season by means of a hydraulic dredge.
2. During falling stages at the end of the highwater season, construct a fill consisting of sand at the base, rock at the top, and gravel between the sand and rock.
3. Construct during the highwater season, a submerged rock fill, on lumber mattress, to an elevation that will cut off flow as the river falls at the beginning of the low water season.

4. Similar to method 3, except that concrete tetrahedrons would be used instead of rock.

5. Similar to method 3, except that layers of willow or frame crib timber mattresses would be used in conjunction with the rock for constructing the submerged dam.

Method 1 was rejected after subsurface explorations disclosed that there was not enough suitable clay available within economic reach of the closure site. Method 2 was rejected after it was disclosed that there was inherent risk of losing the fill as a result of crevasses occurring during construction, which would allow scouring through the coarse materials down into the underlying finer material. Methods 3, 4 and 5, are similar except for the materials used. Because of the comparatively high estimated cost of building and placing concrete tetrahedron blocks, this method was rejected. Although the estimated cost of the rock fill closure, method 3, was slightly more than the cost of method 5, it was selected for making the closure because it was considered the more stable and easier to construct. It would be very difficult and impractical to place the layers of willow or timber mattresses with the high velocities that prevail.

DESIGN OF INITIAL ROCK CLOSURE

The site of the initial rock dam closure was selected because the cross section of the channel is nearly symmetrical about the center line and velocities are relatively uniform at this location (Fig. 4). The rock fill will be constructed during the highwater season. The basic requirements for the submerged rock fill are that its top elevation be just high enough to insure cut off of flow for any stage condition likely to prevail as the Mississippi River stage falls preceding the low water season, and that the rock be large enough to remain stable under the highest velocities that will occur over the dam. From a study of the daily river stages which have occurred since 1931, it was determined that elevation +17 msl was the minimum elevation to which the submerged fill could be constructed with reasonable assurance that it would cut off flow long enough to provide sufficient time to complete the main closure fill. The periods when the stage was below elevation +17 during 1931 to 1956 are shown on Fig. 6.

Studies based on the stage-discharge relationship disclosed that at the time the rock fill will be completed, and during the early part of the falling stage period, the highest velocities over the rocks will occur at the upstream shoulder of the fill. This condition will prevail until the river falls to a stage at which critical depth of flow at the crest is reached. Thereafter a hydraulic jump will occur over the rocks and will move downstream progressively as the stage falls. The highest velocities will occur just upstream of the jump. For the size and shape of fill selected, the computed velocities of flow over the rock fill for various stages in the Mississippi River or various discharges are shown on Fig. 7. The quantity of water that will flow through the rock fill was not included in the analysis of the flow over the rock fill and in determining the stability of the rocks in the fill. It was determined, by computations, that the mean velocity at the crest of the dam will vary from about 9 fps at a stage of about +45, to 15 fps at a stage of about +30. The mean velocity just upstream of the jump will vary from about 15 fps at a stage of +30 to a maximum of 20 fps to 22 fps at a stage of about +24. The required rock size was based on Isbash's formula, modified to provide correction for slope.

For a maximum velocity of 22 fps, and at a unit weight of 115 lbs per cu ft for the rock, the required stone weight is 4,200 lbs. Therefore, derrick, or

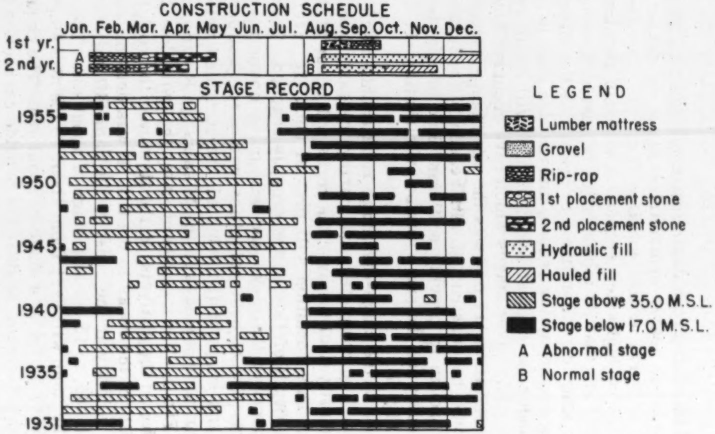


FIG. 6.—CONSTRUCTION SCHEDULE AND RELATION TO EXPERIENCED STAGES

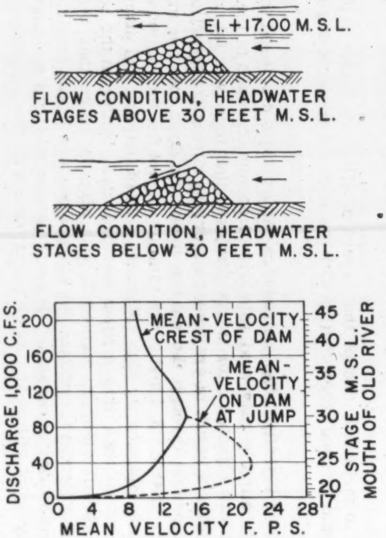


FIG. 7.—FLOW CHARACTERISTICS OVER ROCK FILL

jetty stone, of which at least 65% of the individual stones in each barge load shall be 2 1/4 tons or heavier in weight will be used to make the rock closure. The side slopes of the rock fill will be 1-on-1.25 on the upstream side and 1-on-2 on the downstream side. The stability of the proposed rock fill was tested in a model study at the United States Army Corps of Engineers Waterways Experiment Station at Vicksburg, Mississippi. Based on this study, the proposed slopes are considered necessary to provide a reasonable factor of safety. The study disclosed that a seal blanket on the upstream slope of the rock fill would be necessary to prevent loss of final closure material through the rock. As a result, a seal blanket consisting of a layer of riprap 4 ft thick and a layer of gravel or shell 1.5 ft thick will be placed on the upstream slope of the rock fill, plus an impervious blanket to be pumped and discharged over the upstream face of the fill at the time construction of the main closure dam is commenced. The gradation of the riprap and shell or gravel will be as indicated in Table 1.

TABLE 1.—GRADATION OF UPSTREAM SEAL BLANKET

Size, Inches	Per Cent Passing, by Weight
Riprap (Coarse Layer)	
12	100
7 1/2	60 - 90
5	10 - 40
2 1/2	2
Gravel or Shell (Finer Layer)	
2 1/2	100
1	60 - 90
1/2	30 - 60
1/4	10 - 35
1/20	10

To prevent erosion of the soils in the river bed at the base and downstream of the rock fill, and in the river banks above the elevation of the crest of the rock fill, a lumber mattress and a gravel blanket will be placed over the river bed and on the banks, respectively, as shown on Figs. 8 and 9. A layer of riprap 2 ft thick will be placed over part of the lumber mattress and over the gravel blanket to add additional protection against erosion.

The derrick stone fill will consist of two placements. The first placement will be 5 ft to 6 ft thick, extending up the slopes of the banks from approximate elevations -5 to +35 msl (Figs. 8 and 9). The second placement of derrick stone will be placed over the riprap fill and will be accomplished with a minimum of 18 ft difference between the headwater stage and the elevation of the top of the rock being placed in order to avoid placing stone in high velocity currents. It would have been desirable to locate the rock dam so that it could be incorporated in the downstream portion of the main closure dam, but this was not possible because of hydraulic requirements of the initial closure and the required alignment of the main closure with the bridge across the navigation lock.

DESIGN OF MAIN CLOSURE

The proposed cross section for the main closure dam is shown on Fig. 10. The lower portion of the fill will be composed of sand and the upper part of

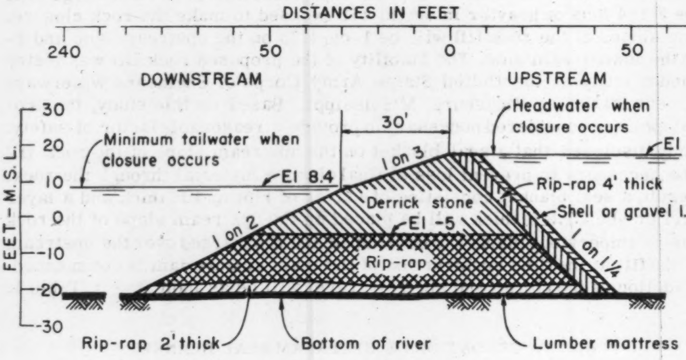

DISTANCES IN FEET
240
50
0
50
DOWNSTREAM
UPSTREAM
30
20
10
0
-10
-20
-30
FEET-M.S.L.
Minimum tailwater when closure occurs
Headwater when closure occurs
30'
El 17.0
El 8.4
1 on 3
Rip-rap 4' thick
Derrick stone
Shell or gravel 1.5' thick
1 on 2
El -5
Rip-rap
1 on 1¼
Rip-rap 2' thick
Bottom of river
Lumber mattress

FIG. 8.—CROSS SECTION OF ROCK FILL DAM

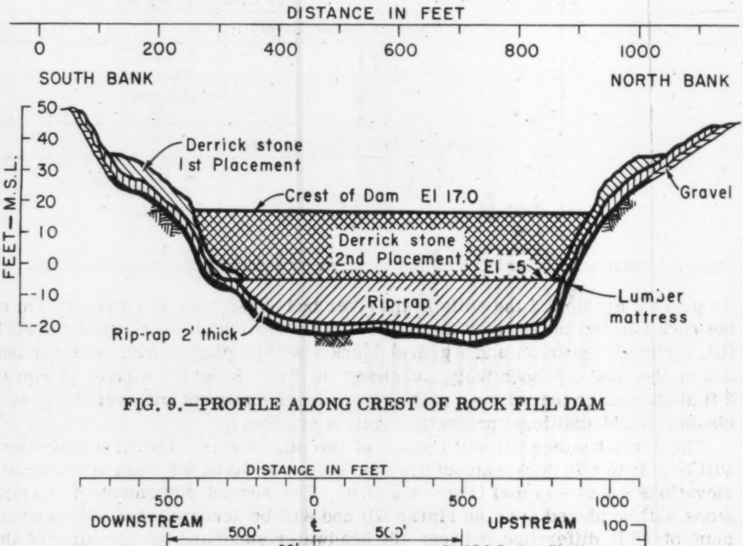
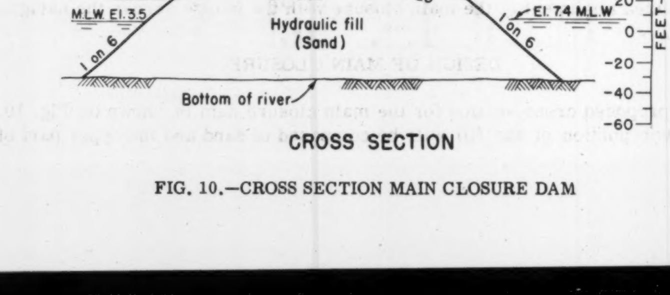

DISTANCE IN FEET
0
200
400
600
800
1000
SOUTH BANK
NORTH BANK
50
40
30
20
10
0
-10
-20
FEET-M.S.L.
Derrick stone
1st Placement
Crest of Dam El 17.0
Gravel
Derrick stone
2nd Placement
El -5
Rip-rap
Lumber mattress
Rip-rap 2' thick

FIG. 9.—PROFILE ALONG CREST OF ROCK FILL DAM


DISTANCE IN FEET
500
0
500
1000
DOWNSTREAM
UPSTREAM
℄
500'
500'
60'
400'
El 68.0
El. 65.0 Project Flood
Hauled fill
1 on 4
1 on 4
5'
Clay blanket
(Hauled fill)
1 on 20
1 on 20
El. 20.0±
El. 45.0±
M.L.W. El. 3.5
El. 7.4 M.L.W
Hydraulic fill
(Sand)
1 on 6
1 on 6
Bottom of river
80
60
40
20
0
-20
-40
-60
FEET-M.S.L.

CROSS SECTION

FIG. 10.—CROSS SECTION MAIN CLOSURE DAM

clays. The 1-on-6 slope for the sand fill below water surface and the 1-on-20 slope above the water surface were based on the natural angle of repose of a previous hydraulic sand fill made across Old River at its entrance with sand obtained from the bed of Old River. The remainder of the main closure fill will consist of clay hauled from an adjacent borrow pit and compacted into a section with 1-on-4 side slopes and a crown 60 ft wide. The exact height to which the sand portion of the fill will be constructed will depend upon the river stage at the time it is placed. A blanket of clay, 5 ft thick, will be placed on the upstream side of the closure over the sand fill exposed above water, and over the sand bar for a distance of 400 ft upstream from the closure center line, for the purpose of reducing seepage through the embankment. A portion of the completed dam along the south one third of the river crossing will be located over a clay stratum which extends in depth to approximate elevation -50 msl.

Based on an assumed failure to the bottom of this clay and using the design shear strength in the clay, and the seepage influence indicated by the flow net, the proposed dam has a minimum factor of safety of 1.5. Assuming a friction angle of 25° with sudden drawdown to elevation +3.5 within the 1-on-6 slope of the sand fill on the downstream side, the factor of safety with respect to shear failure is 1.2. This factor of safety is considered ample because the drawdown conditions assumed are extreme.

SETTLEMENT

It is estimated, based on the consolidation curves, that 2.1 ft of settlement will occur due to consolidation of the foundation. Of this amount 0.8 ft will occur during construction and 1.3 ft after construction. Allowing 1.5 ft for shrinkage and consolidation of the fill after construction the estimated crown settlement after construction will be 2.8 ft. The net grade of the adjacent main-line Mississippi River levee is +64.4 msl. The main closure dam will be built to elevation +68. This allows 3.6 ft for settlement and shrinking.

SEEPAGE

The sand portion of the embankment will not be completely covered with the impervious blanket on the upstream side, and the base of the embankment will be in contact with the substratum sand as disclosed by the borings. Seepage will therefore occur through the exposed sand portion of the fill and through the substratum sand beneath it, as shown by the flow net.

The flow net was based on a permeability of 50×10^{-4} cm per sec for the hydraulic fill sand; horizontal and vertical permeabilities of 400×10^{-4} and 100×10^{-4} cm per sec, respectively, for the substratum sand between elevations -35 and -75, and $1,000 \times 10^{-4}$ and 250×10^{-4} cm per sec, respectively, for the substation sand below elevation -75.

The permeability of the sand fill was determined from the results of permeability tests on composite sand samples from the proposed borrow area, and the permeability of the substratum sand was based on the relationship between permeability and grain size obtained from field pumping tests conducted at Old River lock site. The maximum amount of seepage that may occur on the downstream side of the closure fill, as disclosed by the flow net will be about 559 cu ft per day per linear ft of embankment. Seepage in this quantity could occur only at high river stage when the embankment is first completed, and will progressively decrease in time as fine sediments from the Mississippi River blanket the upstream side of the dam.

BORROW PITS

Materials for the hydraulic fill portion of the main closure will be obtained from the sand bar on the south bank of Old River and from the river bottom on the upstream side of the closure. If required, additional borrow areas on the north bank of Old River will be available, as shown on Fig. 2. The borrow pits for the hydraulic fill will be located a minimum distance of 500 ft from the upstream toe of the main closure fill. Material for constructing the upstream impervious blanket will be obtained from a borrow area located adjacent to the closure on the north side of Old River, as shown on Fig. 2. It is contemplated that materials excavated from the channels for the proposed navigation lock on the south bank of Old River adjacent to the closure will not be available for making the closure. However, should the closure be constructed prior to excavation of the channels, suitable materials from the channel areas will be used insofar as practicable.

Riprap and stone are not available locally and will have to be obtained elsewhere and transported to the site of the work in barges.

METHODS OF CONSTRUCTION

Closure of Old River will be accomplished in three phases of construction; preparation of the initial closure area to prevent erosion, construction of the initial closure (rock fill dam), and construction of the main closure (hydraulic fill and hauled fill).

The dumping of all materials in the water will be controlled so that the materials will settle as near as possible within the predetermined location on the river bottom. Control will be exercised by dumping the materials at a trial distance upstream from the desired location, depending on the velocity of the water and depth of the river. Soundings will be taken to determine the actual location of the dumped material on the bottom of the river, then the distance upstream to the dumping point will be adjusted accordingly.

Preparation Of Initial Closure Area (Rock Fill Dam).—During the low water season in the year preceding the construction of the main closure, an area consisting of a strip 300 ft wide extending across Old River will be prepared as a base for the initial rock closure. As shown on Fig. 2, the upstream boundary of the area will be 60 ft upstream from hydrographic range 49 and the downstream boundary will be 240 ft downstream from this range.

The upper banks of Old River within the base area will be graded to a 1-on-3 or flatter slope. A continuous lumber mattress will be placed over the base area below water. The upper banks above water elevation will be paved with a 12-in. layer of gravel covered with 24-in. of riprap. The portion of the revetted area 40 ft upstream and 90 ft downstream from hydrographic range 49 and below the water surface extending across Old River, will be covered with 24-in. of riprap to form a base upon which to place the riprap and larger stone. The riprap will be delivered to the site on barges and will be handled from barges by draglines at a minimum rate of 1,000 tons per day. The rate is based on unloading two barges at a time by dragline. Estimated time required to complete the preparation of the base is 50 days. This work is scheduled to be done during the months of August and September of the first construction year, but may be done in any suitable period, preceding placement of stone.

Construction Of Initial Closure (Rock Fill Dam).—The initial closure will be accomplished by placements of riprap and stone during the high water season following the preparation of the base (Figs. 8 and 9). Placement of the stone will be interrupted or discontinued when the difference between the headwater elevation and the elevation of the top of the stone being placed is 18 ft or less, as would have occurred in the years 1931 and 1954 (Figs. 11 and 12). The first placement is scheduled to insure completion by the 23rd of March, which is the limiting time for commencement of the second placement. This date was established from hydrographs depicting the lowest high water seasons during the period 1931 through 1956 (Fig. 6). The placement of riprap and stone is scheduled to be accomplished at the same rate. The commencement date of the riprap placement may be advanced, however, if desired to January 1 and the placement rate reduced accordingly. The first and second placements of stone will be made as scheduled or faster. The seal blanket will be constructed when the head over the rock fill dam has been reduced to about 3 ft and a continuous fall in stage is predicted.

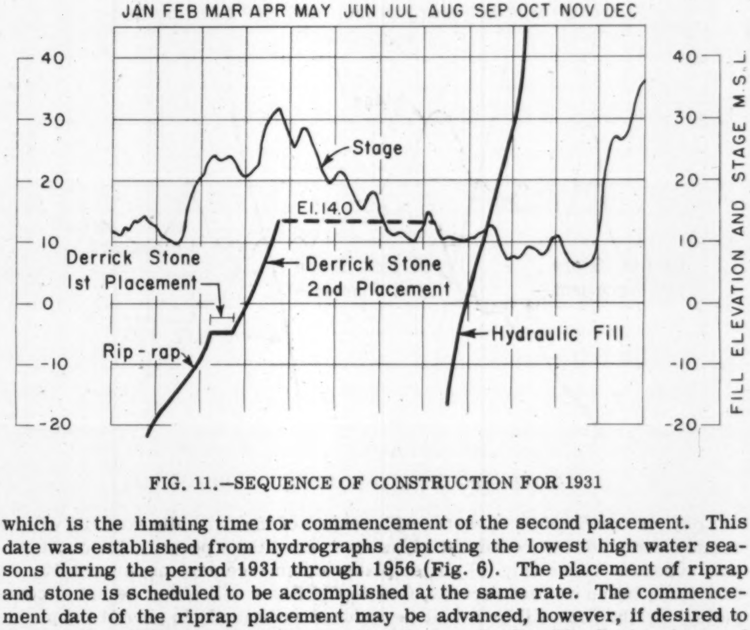


FIG. 11.—SEQUENCE OF CONSTRUCTION FOR 1931

The riprap will be placed in horizontal layers 3 ft to 4 ft thick to elevation -5 msl. The first placement of derrick stone will consist of a layer 5 ft to 6 ft thick of 2 1/4 ton stone extending up each bank from elevation -5 to elevation +35 msl (Fig. 9). The second placement will be accomplished by progressively building up the cross section above the riprap in horizontal layers about one rock thick to attain an upstream slope of 1-on-1.25, a crown slope of 1-on-3, and a downstream slope of 1-on-2 (Fig. 8). Placing of the 2 1/4 ton stone will continue until the initial rock fill is completed to elevation +17 msl or to the maximum elevation reached before termination of construction due to the limiting depth of water flowing over the rock. A review of the hydrographs, 1931-1956, shows that the rock dam could have been constructed to elevation +17 msl in 19 of the 26 yr. In the other 7 yr the initial closure could not have been constructed to elevation +17, but the low water following the placement of the stone would have been low enough to insure final closure.

To allow for settlement, and stone and riprap placed out of section, the quantities of stone and riprap required were computed on the basis of 1.75 tons per cu yd rather than 1.35 tons normally used for dry land construction. In addition, the quantities were increased by about 13 1/2% to allow for other contingencies. The riprap and 2 1/4 ton derrick stone will be delivered to the site on barges and placed in position in the river at a minimum rate of 1,700 tons per day. Estimated time required to place the stone for the initial closure is 86 days.

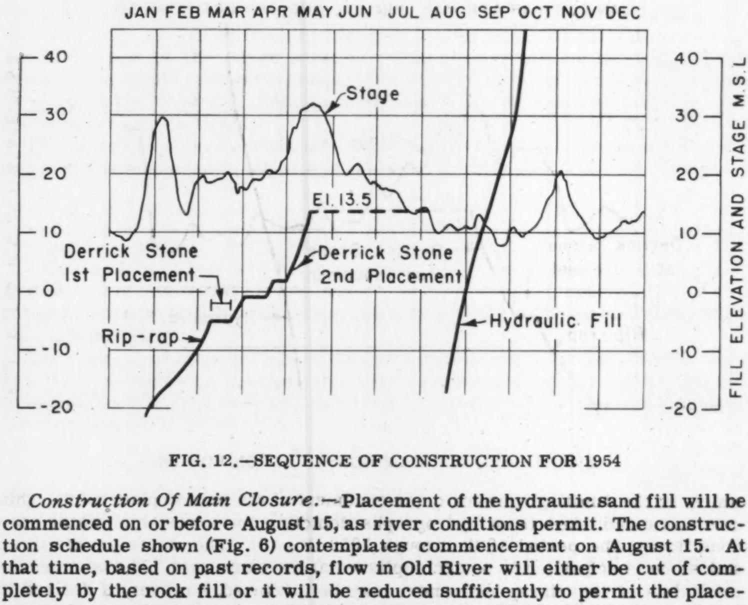


FIG. 12.—SEQUENCE OF CONSTRUCTION FOR 1954

Construction Of Main Closure.—Placement of the hydraulic sand fill will be commenced on or before August 15, as river conditions permit. The construction schedule shown (Fig. 6) contemplates commencement on August 15. At that time, based on past records, flow in Old River will either be cut of completely by the rock fill or it will be reduced sufficiently to permit the placement of sand without appreciable loss. The sand will be placed at a rate that will insure completion of the portion of the closure cross section up to elevation 0 msl in 15 days, to elevation +28 in 45 days, and up to elevation +35 by October 6.

Based on the assumption that 50% of the material pumped by the dredges will remain in the planned cross section, and an average rate of pumping for one dredge (including allowance for breakdowns) of 35,000 cu yd per day, two dredges will be required to accomplish the hydraulic fill work according to the afore-mentioned schedule. The relationship of the sequence of construction of the hydraulic fill to average hydrographs is shown on Fig. 13.

A study of actual hydrographs shows that in most cases (22 yr out of 26 yr) the schedule could have been followed without interruption. The year 1950 (Fig. 14) represents the worst of four cases where the schedule could be disrupted and delay could result because of prevailing abnormal high stages during the months of September and October. The year 1945 (Fig. 15) represents the only case in which a rapid rise and abnormal high water during the low water season would have almost overtopped the fill.

Based on the records of the past 26 yr (since 1933), the construction sequence shown on Fig. 6 allows sufficient time to complete the hydraulic sand portion of the closure fill with two dredges. It will require about 56 days for pumping the sand fill if normal conditions prevail, and 94 days if the most adverse experienced in the past reoccur at the time the fill is being pumped.

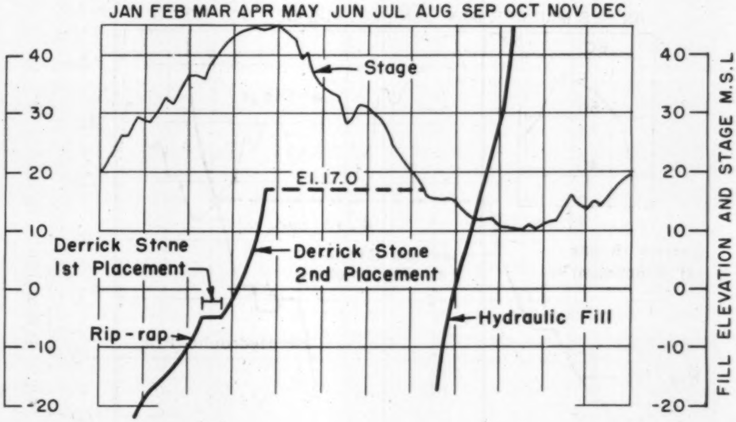


FIG. 13.—1935-1954, AVERAGE SEQUENCE OF CONSTRUCTION

For a condition similar to that which prevailed in 1950, a portion of the partial sand fill would be washed out during construction and the dredges would have to stand by idle for a period of about 38 days. In constructing the hydraulic fill the sand in the river bottom will be used first and the sand in the bar reserved for the final portion of the fill. The height to which the hydraulic fill will be built will be limited by the stage prevailing at the time the fill is completed. For estimating purposes this height was assumed to be at elevation +45.

Hauled Fill.—Immediately following the completion of the sand hydraulic fill, hauling equipment will commence placing and semi-compacting the hauled fill portion of the main closure and the impervious blanket on top of the sand fill and sand bar. The estimated time of completing the hauled fill including the impervious blanket is 45 days. An alternate plan for the hauled fill embankment is provided in the event that adverse weather or river stages make it impractical to use hauling equipment. This plan contemplates the use of an all-hydraulic fill.

The basic requirement of the alternate construction plan is to complete a stable embankment across Old River to elevation +68 msl by hydraulic method during the low water season. The fill material will be sand except for an impervious clay blanket to be placed on the upstream slope. Based on a previous hydraulic sand fill made in Old River run-off from the dredge discharge over slopes steeper than about 1-on-20 will result in serious erosion of the fill.

It is proposed to construct the sand fill in four phases as described below and as shown by the cross sections on the sketch, Fig. 16. A clay blanket will be added on the upstream side as Phase V. In Phase I, the embankment will be placed as planned in the design memorandum to about elevation +45 msl with side slopes of about 1-on-20 to water surface and 1-on-6 below water surface. In Phase II, retaining dikes located about 200 ft each side of and parallel to the embankment centerline will be constructed with sand obtained from the center of the embankment, as shown on the sketch. These retaining dikes will be constructed with dragline equipment and will have slopes of about 1-on-4 with crown elevations at least +50 msl. The dikes will be constructed continuous from one bank of the river to the other and will be tied into the river banks at the ends.

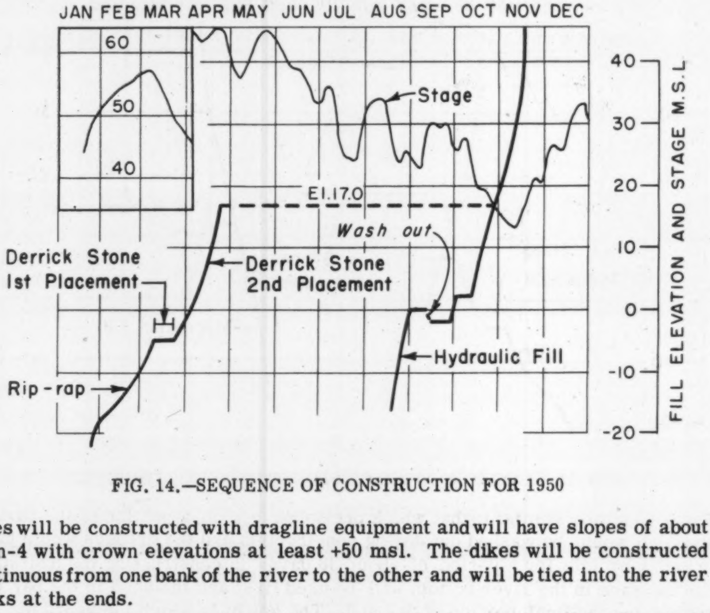


FIG. 14.—SEQUENCE OF CONSTRUCTION FOR 1950

A ditch will be excavated on the north river bank to drain-off dredge waste water. The ditch will be located generally along the embankment centerline where it commences and will be normal to the river for a distance about 200 ft from the river. From there, it will circle along the river bank and connect back with the river about the embankment. The bottom of the ditch will be at about elevation +42 msl, and will drain off the waste dredge water between the retaining dikes but will also permit water to be impounded on the hydraulic fill between the retaining dikes without over topping the dikes.

In Phase III, a second lift hydraulic fill will be placed between the retaining dikes. This fill will be constructed mostly in impounded water to about elevation +55 msl with ultimate slopes of 1-on-20. The waste water run-off during construction of Phase III will either flow through the ditch in the north river bank or seep through the embankment. In either event the disposal of the waste water will not result in erosion of the fill. The hydraulic fill in Phase III will be constructed from the south bank progressively across the river.

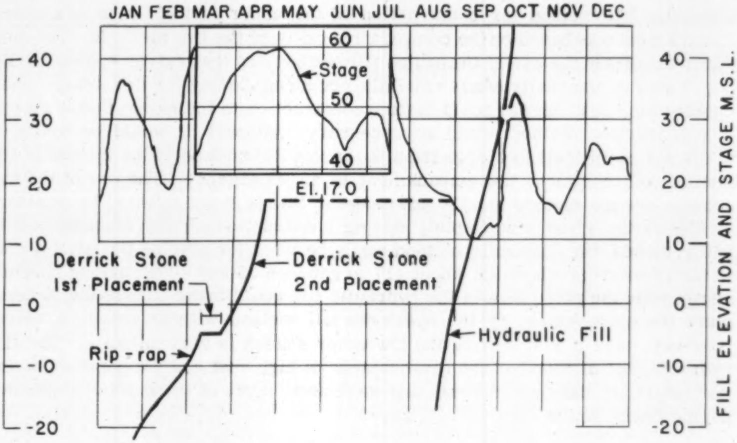


FIG. 15.—SEQUENCE OF CONSTRUCTION FOR 1945

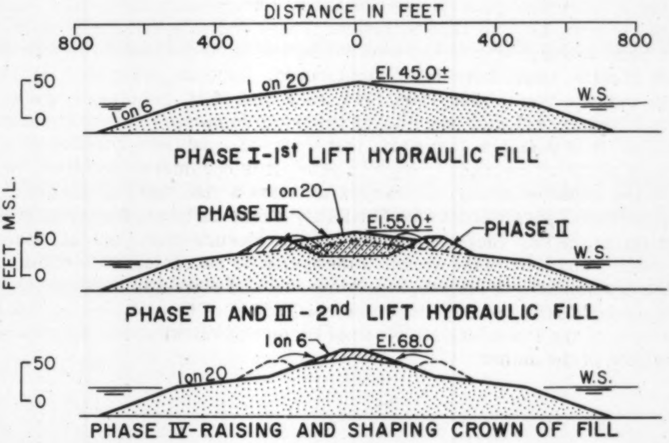

DISTANCE IN FEET
800
400
0
400
800
50
0
1 on 6
1 on 20
El. 45.0±
W.S.
PHASE I-1st LIFT HYDRAULIC FILL
FEET M.S.L.
1 on 20
PHASE III
El. 55.0±
PHASE II
W.S.
PHASE II AND III - 2nd LIFT HYDRAULIC FILL
1 on 6
El. 68.0
1 on 20
W.S.
PHASE IV-RAISING AND SHAPING CROWN OF FILL

FIG. 16.—ALTERNATE HYDRAULIC CLOSURE

In Phase IV, the upper part of the embankment will be raised and shaped by dragline to a crown width of 60 ft at elevation +68 msl with side slope of 1-on-6, as shown on the sketch. The impervious blanket will be placed, in Phase V. This blanket will consist of clay obtained from the clay stratum in the north bank of Old River between approximate elevations +25 and +50 msl during high water stage, or from the clay stratum beneath the sand bar near the south bank during low water. A 20-in., or smaller, dredge will be used for placing the blanket. Use of a larger dredge will wash away the sand fill.

The use of the hauled fill method proposed in the basic design in lieu of the hydraulic fill method for construction of the upper part of the main closure embankment was based on the comparative costs of the two methods. The only disadvantages to the use of the hauled fill method are that extreme, abnormally high water or unusually heavy rainfalls occurring during the dry season when construction is underway could disrupt construction to the extent that a charge in construction method would be necessary. Although it would be costly, it would not be difficult to change from the hauled fill method to the hydraulic fill method and complete the embankment in an emergency. The use of the hydraulic dredge method has the advantage in that it is not affected by weather.

Should the river rise rapidly during construction as the embankment is being raised, the hydraulic method would be ideal for placing the fill. A rise in the river from elevation about +17 to between 35 and 40 occurring concurrently with the placement of the hydraulic fill would make it possible to complete the embankment by the hydraulic fill method without retaining dikes. However, such a rise during the low water season is very unlikely. The all-hydraulic fill method has the disadvantage of high cost and the possibility that the retaining dikes could break and wash away some of the main embankment during construction.

CONCLUSIONS

The construction plan for the Old River closure is based on 26 yr of river stage records, 1931 to 1956 inclusive. These indicate that the closure could have been accomplished in any year during this period, although, in four cases, some trouble would have been experienced. Records prior to 1931 (1871 to 1930), indicate that in 1905, 1906, 1915, 1926, and 1927, the river stage was not below elevation +17 msl long enough to permit completing the main closure as planned. It is possible, therefore, that adverse conditions could exist so that closure could not be made in a given year. It is considered practical, preferable, and sound engineering practice to follow a plan that will assure against all conceivable possibilities with flexibility where dictated. Recognition should be given to the fact that any plan for a river closure must take full account of the geology and soils of the area and the hydraulic characteristics of the stream in the development of the proper engineering and economic solution.

Construction of the Old River Control Project is being conducted under the direction of the President, Mississippi River Commission, and the general supervision of the author.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3171

SHARK RIVER INLET SAND BY-PASSING PROJECT

By W. Mack Angas[1]

SYNOPSIS

The by-passing of sand across Shark River Inlet on the New Jersey Coast appears to be an effective method of nourishing and restoring a starved beach at the down-drift side of the Inlet. The by-passing operation, which was undertaken as a full scale experiment, is considered successful because:

1. The sand by-passed across the Inlet is coarser, better graded, and has made a beach fill that is more stable than beach fills made elsewhere on the New Jersey Coast with fine sands such as might have been obtained by dredging in Shark River.

2. The by-passed sand makes a more attractive beach than sands usually obtained by dredging in inland waters inasmuch as it is freer of objectionable shell fragments, clay balls, and silt.

3. Sand obtained from the accretion at the up-drift side of the Inlet has proved cheaper than sands obtained from other sources when comparative costs are based on the unit price of sand that stays in place on the beach.

4. The removal of the excess accretion at the up-drift side of the Inlet is benefiting the community facing the up-drift beach in three ways: It restores to utility much of the length of a "fishing pier" which was being made useless by the development of dry beach under it; the reduction of the beach berm to an optimum width of from 100 ft to 200 ft has been found desirable; and it is anticipated that the removal of the excess accretion will facilitate the maintenance of the navigation channel of Shark River Inlet.

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INTRODUCTION

Shark River Inlet lies in a popular resort section of the northern New Jersey Coast, where the progressive erosion of the beaches has long been a matter of grave concern. In 1954 the Corps of Engineers, representing the Federal Government, and the Department of Conservation and Economic Development of the State of New Jersey, representing the State, completed a cooperative study of the control of the erosion of that part of the Atlantic Coast of the State lying between Sandy Hook and Barnegat Inlet which points Shark River Inlet lies.

This cooperative study led to the preparation of a report, approved by the Beach Erosion Board, whose findings and recommendations may be summarized as follows:

The 51 miles of the New Jersey Coast lying between Sandy Hook and Barnegat Inlet (Fig. 1) consist of an 11-mile stretch of barrier beach which reaches to Monmouth Beach, followed by 19 miles of headland terminating at Bay Head and finally 21 more miles of barrier beach the south end of which is Barnegat Inlet. There are littoral movements of sand northward to Sandy Hook and southward to Barnegat Inlet from a nodal point about 40 miles south of Sandy Hook and 11 miles north of Barnegat Inlet. Practically the entire 51 miles of the coastline has long been eroding progressively (Figs. 2, 3, and 4). The bulkhead shown in Fig. 2 failed to stop the progressive loss of the upland. The beach shown in Fig. 3 has gone and the upland is starting to go. During the past 50 or 60 yr this progressive erosion has been accelerated on the down-drift sides of various man-made obstructions to littoral movements of sand. These obstructions consist of jetties built at the mouths of the Manasquan and Shark Rivers to stabilize the positions of the river mouths and facilitate the maintenance of satisfactory channel depths, and groins constructed for the prevention or control of erosion.

The remedial measures recommended by the report (Fig. 5), are the restoration of starved and eroded beaches by beach filling to provide berms at least 100 ft wide and 10 ft above MLW, the construction of a few additional groins where these are considered necessary to prevent rapid reduction in the width of the widened berms by littoral movement of sand, and the regular and systematic nourishment of the beaches by establishing and periodically replenishing feeder beaches or stock piles from which sand would be distributed by previously mentioned littoral movements.

The Department of Conservation and Economic Development of the State of New Jersey, the agency responsible for the maintenance and protection of shores and beaches, is in general agreement with the recommendations of the report. Shortage of funds, however, has prevented the immediate full implementation of these recommendations. The policy that has been, and is being, followed, is to spend available funds, insofar as possible, for the accomplishment of projects compatible with the recommendations of the report and to adopt such measures as are considered appropriate when an emergency is created by a sudden localized erosion.

As a means of nourishing a starved beach and possibly stock piling sand at the down-drift side of an inlet that has a large unwanted accretion at its updrift side, sand by-passing suggested itself for several reasons. The first of these is that the State has several inlets protected by jetties, where, at the updrift side, large and unnecessary accretions have formed (Figs. 6, 7, and 8). In Fig. 6 the jetties interrupt a southwestward littoral movement of sand toward Cape May, in Fig. 7 northward littoral movement of sand is interrupted, and in Fig. 8 the starved and eroded condition of the beaches north of the inlet

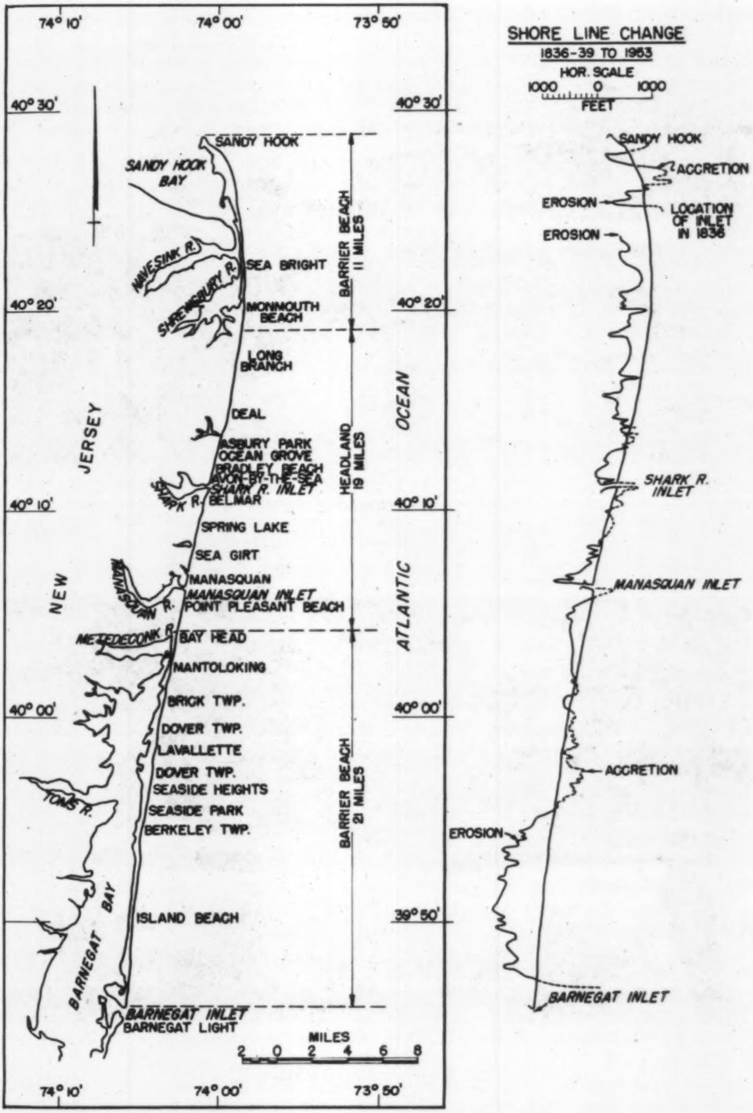

74° 10'
74° 00'
73° 50'
40° 30'
40° 20'
40° 10'
40° 00'
39° 50'
SANDY HOOK
SANDY HOOK BAY
NAVESINK R.
SHREWSBURY R.
SEA BRIGHT
MONMOUTH BEACH
LONG BRANCH
DEAL
ASBURY PARK
OCEAN GROVE
BRADLEY BEACH
AVON-BY-THE-SEA
SHARK R. INLET
SHARK R.
BELMAR
SPRING LAKE
SEA GIRT
MANASQUAN
MANASQUAN R.
MANASQUAN INLET
POINT PLEASANT BEACH
METEDECONK R.
BAY HEAD
MANTOLOKING
BRICK TWP.
DOVER TWP.
LAVALLETTE
DOVER TWP.
SEASIDE HEIGHTS
SEASIDE PARK
BERKELEY TWP.
TOMS R.
ISLAND BEACH
BARNEGAT BAY
BARNEGAT INLET
BARNEGAT LIGHT
NEW JERSEY
ATLANTIC OCEAN
BARRIER BEACH 11 MILES
HEADLAND 19 MILES
BARRIER BEACH 21 MILES
MILES
2 0 2 4 6 8
SHORE LINE CHANGE
1836-39 TO 1953
HOR. SCALE
1000 0 1000
FEET
SANDY HOOK
ACCRETION
EROSION
LOCATION OF INLET IN 1836
EROSION
SHARK R. INLET
MANASQUAN INLET
ACCRETION
EROSION
BARNEGAT INLET

FIG. 1.—NEW JERSEY COAST, SANDY HOOK TO BARNEGAT INLET

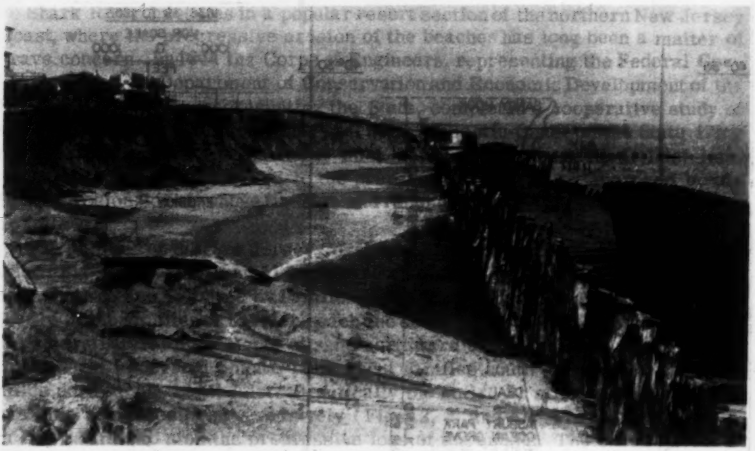


FIG. 2.—HEADLAND EROSION

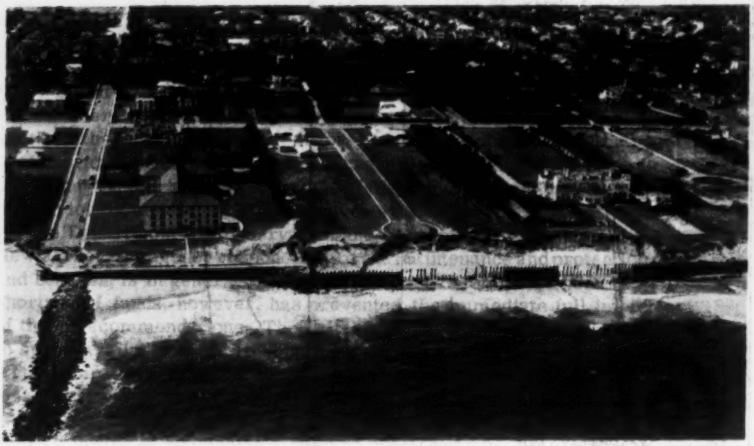


FIG. 3.—HEADLAND EROSION

is evident. At all these locations there is an unfortunate lack of clean, coarse sand which could be obtained for beach filling by dredging in a nearby inland waterway. Finally, though beach filling with fine sands obtainable by suction dredging in nearby inland waters has proved reasonably effective in a number of locations, it was considered worth-while to try as an alternative the restoration of a starved down-drift beach by by-passing sand from an up-drift accretion.

SHARK RIVER INLET

Shark River Inlet (Fig. 8) was chosen as the site of the experimental by-passing operation because:

1. The beach on the down-drift or north side of Shark River Inlet is one of the most heavily eroded, highly developed ocean frontages in New Jersey.

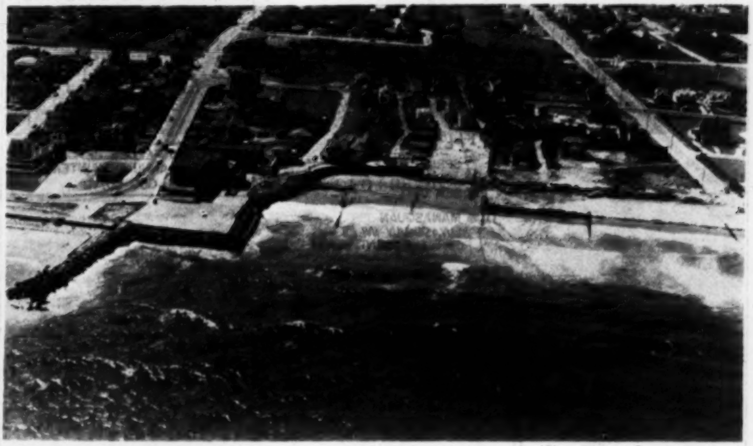
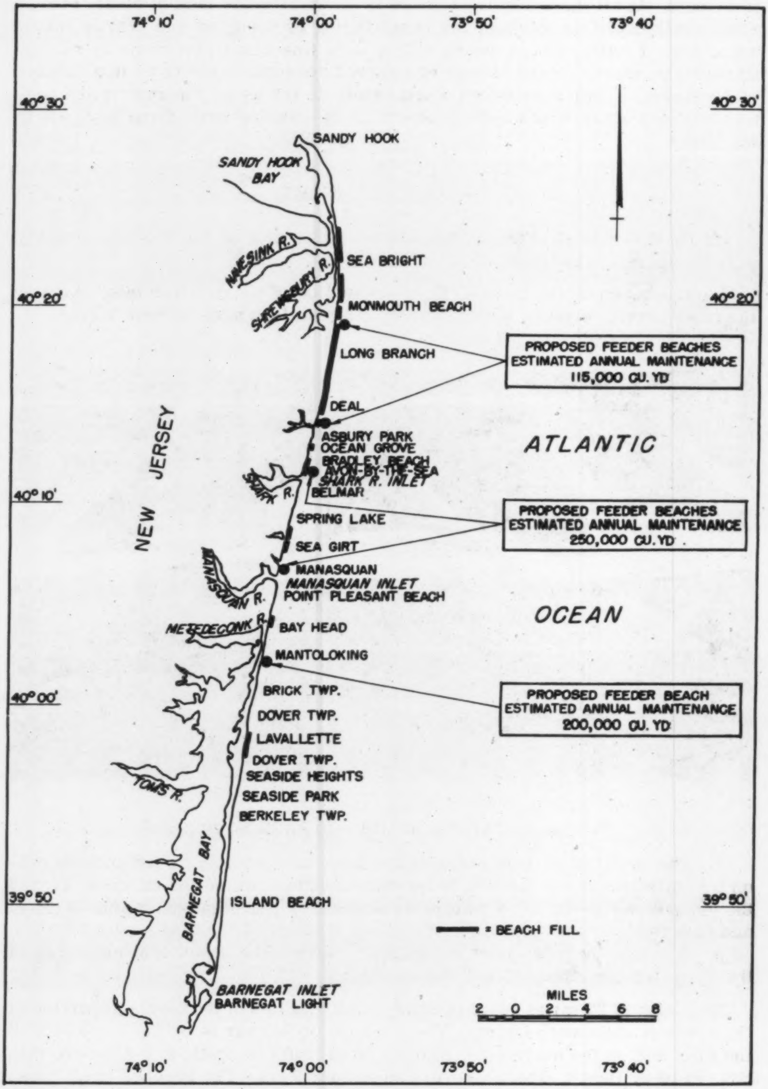


FIG. 4.—A STARVED BEACH AND ERODING UPLAND

2. The beaches on both sides of the Inlet are publicly owned and the municipal officials and engineers of the communities owning the beaches favored the experiment and agreed to furnish necessary permissions, rights of ways and easements.

3. A bridge over the Inlet immediately behind the beach facilitated one of the by-passing methods under consideration.

The Inlet is 20 miles south of Sandy Hook and lies in the headland portion of the coast of Monmouth County. The Borough of Belmar is on the south side of the Inlet and on the north side, reading from south to north, are Avon-by-the-Sea, Bradley Beach, Ocean Grove, and Asbury Park. The beaches of all these communities are publicly owned and are heavily used during the summer season for recreational purposes. Each of the municipalities has, with financial aid from the State of New Jersey, constructed rock or timber groins along its beach front to retard loss of sand and give some measure of protection from storm damage to roads and structures on the upland. The system of groins is


74° 10'
74° 00'
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73° 40'
40° 30'
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SANDY HOOK
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MANASQUAN INLET
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POINT PLEASANT BEACH
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BAY HEAD
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BRICK TWP.
DOVER TWP.
LAVALLETTE
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SEASIDE HEIGHTS
SEASIDE PARK
BERKELEY TWP.
TOMS R.
BARNEGAT BAY
ISLAND BEACH
BARNEGAT INLET
BARNEGAT LIGHT
NEW JERSEY
ATLANTIC
OCEAN
PROPOSED FEEDER BEACHES ESTIMATED ANNUAL MAINTENANCE 115,000 CU. YD
PROPOSED FEEDER BEACHES ESTIMATED ANNUAL MAINTENANCE 250,000 CU. YD
PROPOSED FEEDER BEACH ESTIMATED ANNUAL MAINTENANCE 200,000 CU. YD
= BEACH FILL
MILES
2 0 2 4 6 8

FIG. 5.—PROPOSED PLAN FOR BEACH RESTORATION AND MAINTENANCE

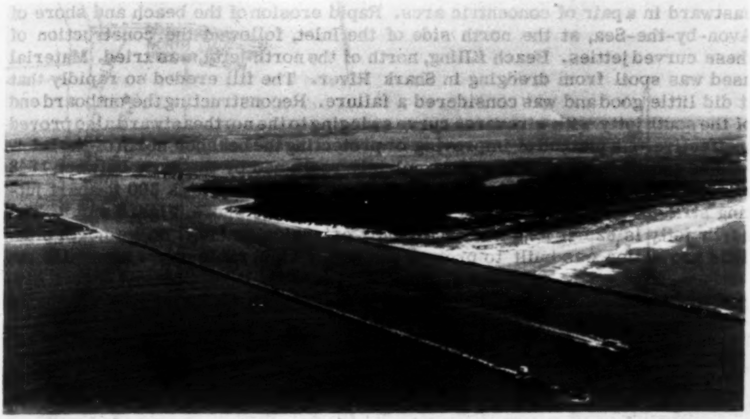

FIG. 6.—COLD SPRING INLET

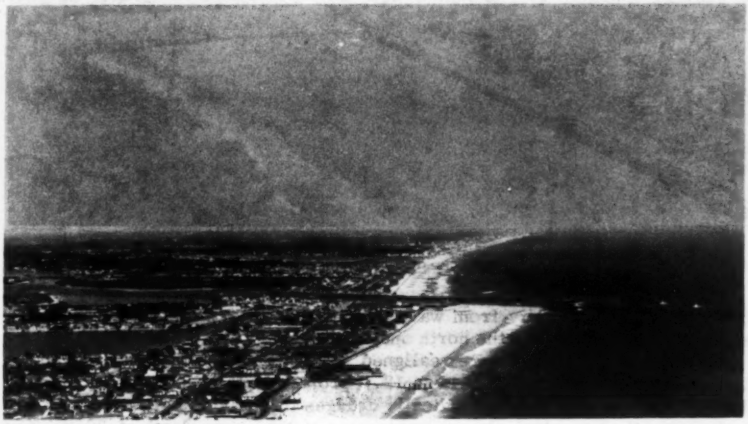

FIG. 7.—MANASQUAN INLET

considered complete at this time and no further groin construction is contemplated within the areas under discussion.

Shark River Inlet is a federally maintained navigation inlet stabilized by jetties. The first jetties, built in 1915, were of rock and curved to the southeastward in a pair of concentric arcs. Rapid erosion of the beach and shore of Avon-by-the-Sea, at the north side of the Inlet, followed the construction of these curved jetties. Beach filling, north of the north jetty, was tried. Material used was spoil from dredging in Shark River. The fill eroded so rapidly that it did little good and was considered a failure. Reconstructing the outboard end of the south jetty with a reverse curve swinging to the northeastward also proved ineffective as a method of improving or protecting the beaches north of the Inlet.

The jetties were rebuilt and completely realigned by the State between 1948 and 1951. They now consist of a pair of parallel rock jetties 300 ft apart jutting straight out into the ocean at right angles to the shore, Figs. 8 and 9. The north jetty is 625 ft long and has a short extension or "lug" on its north side. This projection was built to protect the highly vulnerable shore alongside the landward end of the jetty from wave action during northeasterly gales. The south jetty is longer than the north one, having a total length of 950 ft.

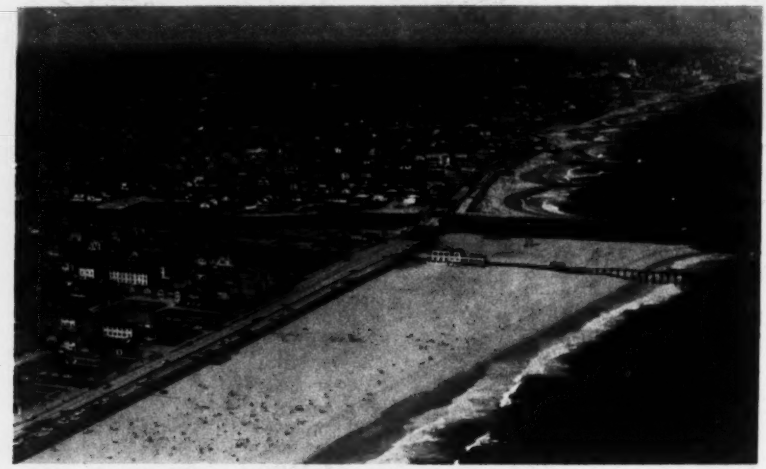

FIG. 8.—SHARK RIVER INLET

At the time the jetties were realigned, the Federal government dredged the Inlet and Shark River to provide a 12-ft project depth to the Belmar marina. The Federal government periodically dredges the channel between the jetties and across the bar which forms to seaward of them to a depth of 20 ft. The dredging is done by a small seagoing hopper dredge, the material being dumped at sea in deep water.

TIDES, WAVES, AND LITTORAL DRIFT

The normal range of the tide at the Inlet is 4 ft with spring ranges of 5 ft. Shark River has a small flow, is not subject to heavy flash floods, and brings down negligible quantities of sand and sediment to the beaches. Wave heights

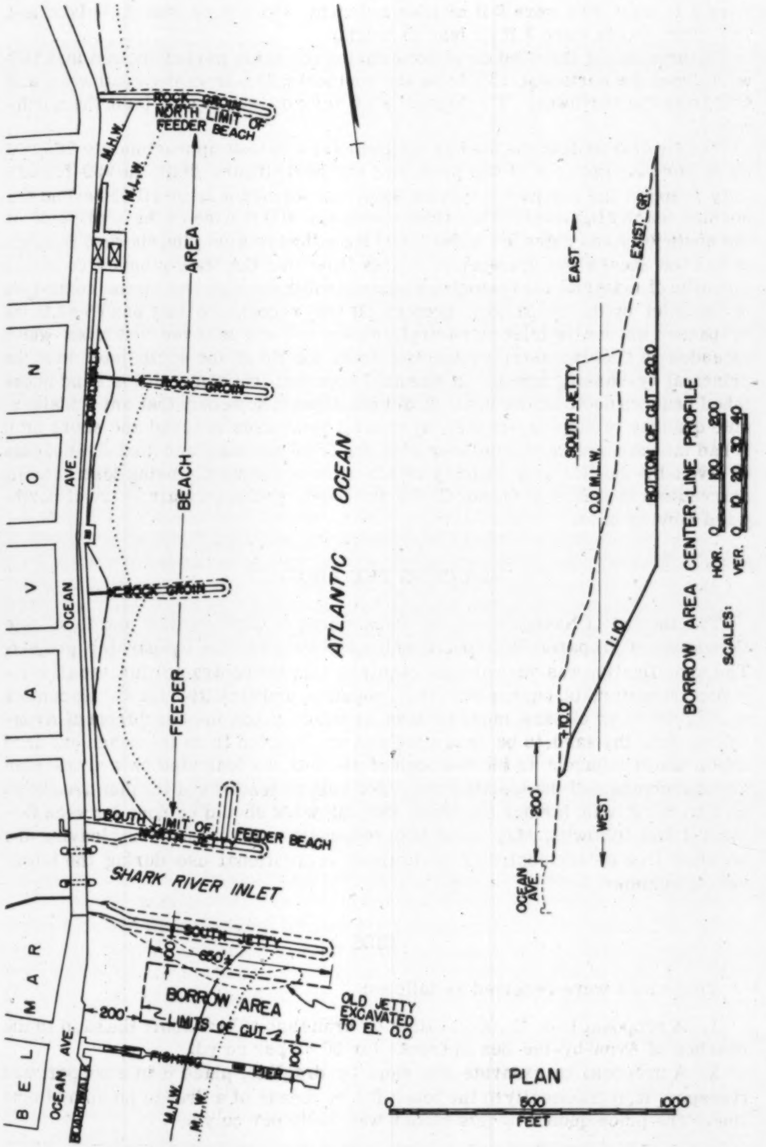
B E L M A R
OCEAN AVE.
BOARDWALK
M.H.W.
M.L.W.
200'
LIMITS OF CUT -2
FISHING PIER
100'
BORROW AREA
OLD JETTY EXCAVATED TO EL. 0.0
SOUTH JETTY
650'±
100'
SHARK RIVER INLET
NORTH JETTY
SOUTH LIMIT OF FEEDER BEACH
A V O N
OCEAN AVE.
BOARDWALK
ROCK GROIN
ROCK GROIN
ROCK GROIN
FEEDER BEACH AREA
NORTH LIMIT OF FEEDER BEACH
M.H.W.
M.L.W.
ATLANTIC OCEAN
PLAN
0 500 1000
FEET
OCEAN AVE.
200'
WEST
EAST
+10.0
1:10
SOUTH JETTY
0.0 M.L.W.
BOTTOM OF CUT -20.0
EXIST. GR.
BORROW AREA CENTER-LINE PROFILE
SCALES: HOR. 0 100 200
VER. 0 10 20 30 40

FIG. 9.—CONTRACT PLAN FOR SAND BY-PASSING PROJECT

were observed from November 1958 to April 1959. The largest waves observed were 7 ft high, 99% were 6 ft or less in height, 93% 4 ft or less in height, and 78% of all waves were 2 ft or less in height.

Observation of the wind directions during the same period showed that 19% were from the northeast, 13% from the southeast, 22% from the southwest, and 46% from the northwest. The highest wind velocity was 35 mph from the northwest.

The littoral drift at the Inlet is northward at a rate of approximately 265,000 cu yd per yr. Because of the prevalent northerly littoral drift, the 950-ft south jetty trapped the northward moving sand and formed a large fillet beyond the normal beach alignment. This fillet was some 400 ft wide at the south side of the south jetty and extended about 1,400 ft southward along the shore of Belmar. It was the excess beach material in this fillet that the State planned to use as a source of material for restoring and nourishing the beaches on the north side of the Inlet by the by-passing project. It was recognized that sand was being by-passed across the Inlet by natural processes[2] and believed that a bar which extended in a northeasterly direction from the tip of the south jetty was the principal by-passing agency. It was also apparent that some of this sand nourished beaches north of the Inlet. It did not appear, however, that any considerable quantity of sand by-passed by natural processes reached the shore until it had moved northward a number of miles from the Inlet and that the beaches of Avon-by-the-Sea and Bradley Beach were consequently being destructively starved and the beach at Ocean Grove was hardly getting its fair share of northward moving sand.

PLANNING THE PROJECT

The Bureau of Navigation of the Department of Conservation and Economic Development prepared the specification and plan for the by-passing project. The specification was very simple requiring that the contractor furnish all construction materials, equipment, labor, supplies, and facilities for the placement of 250,000 cu yd of sand, more or less, as beach fill on the beachfront of Avon-by-the-Sea, the sand to be excavated and transported from the beach and land under water adjacent to the Borough of Belmar, as indicated on a single plan which accompanied the specification. The salient features of the plan are shown on Fig. 9. It was further specified that the work should be done between October 1 and following May 31 of two successive winter seasons, leaving the beaches free of any obstructions to their recreational use during the intervening summer.

BIDS

Three bids were received as follows:

1. A proposal to do the excavation by crane and to transport the sand to the beaches of Avon-by-the-Sea in trucks for $0.88 per cu yd.
2. A proposal to excavate the sand by dragline, place it in a hopper, and transport it hydraulically to the beach fill by means of a dredge pump and pipe line. The price quoted by this bidder was $1.30 per cu yd.

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[2] "Natural By-Passing of Sand at Coastal Inlets," by Per Bruun and F. Gerristsen, Proceedings, ASCE, Vol. 85, No. 4, December, 1959, pp. 75-107.

3. A proposal to do the work with a hydraulic dredge and pipe line for a price of $1.42 per cu yd.

The contract was awarded on the first proposal at the bid price of $0.88 per cu yd for excavating the sand with a crane and bucket and transporting it by truck.

EXECUTION OF THE PROJECT

During late September and October 1958 the contractor built a trestle in the borrow area reaching from Ocean Avenue to a little beyond the low water mark. The trestle was about 425 ft long. He also built three 200-ft trestles north of the Inlet from which to dump sand on the beaches to be filled. The locations of these trestles are shown on Fig. 9. Two of the trestles were in Avon-by-the-Sea and the third on the line dividing Avon-by-the-Sea from Bradley Beach.

On about November 1, the contractor commenced excavating sand from the borrow area with a crane and 2-1/2 cu yd clam shell bucket. A bulldozer was used on the beach to push sand to the crane (Fig. 10). Transportation of the sand to Avon-by-the-Sea was done with two 20-cu-yd trucks and one 10-yd truck. The trestles on which the sand was delivered to the beaches north of the Inlet reached a little beyond low water (Fig. 11), and the contractor proposed to leave the distribution and spreading of the sand from the dumping points at the ends of the trestles to wave action. This would be permitted if wave action proved effective. It did in fact prove highly effective and no spreading of the fill with bulldozers proved necessary during the first winter's operation, though it was necessary to extend the trestles as the beach widened.

The rate at which sand was moved by littoral drift into the borrow area during the winter season of northwest winds was disappointing. On April 17, 1959, the supply of sand that could be reached by the contractor's equipment was insufficient to justify further work at the time and excavation was accordingly stopped. The amount of sand that had been placed on the beaches north of the Inlet was, however, considerable and had effected a great improvement in the condition of the beaches (Figs. 12 and 13). To be specific, the amounts of fill placed from the trestles were:

Trestle 1 —	38,000	cu yd, truck measure.
Trestle 2 —	48,000	cu yd, truck measure.
Trestle 3 —	51,000	cu yd, truck measure.
Total	137,000	cu yd, truck measure.

The trestles were then removed and beaches cleaned up for summer use.

During the summer of 1959 the normal northward littoral movement of sand took place and the excavated borrow area filled rapidly. On October 15, 1959, the contractor was directed to reactivate the project and he at once started to replace the trestle in the borrow area and to reconstruct trestle 1 and 2. Trestle 3 was not reconstructed but two new trestles, 4 and 5 (Fig. 15), were built. Trestle 4 is in Bradley Beach between the northernmost groin in Avon-by-the-Sea and the first one in Bradley Beach. Trestle 5 is north of the first on the Bradley Beach groins. Trestles 4 and 5 do not reach as far into the water as do trestles 1 and 2, and it has been necessary to spread some of the sand dumped from them by means of a bulldozer. Excavation and placement of sand during the second season started on October 22, 1959 and to date 88,000 cu yd have been by-passed this winter or 225,000 cu yd since the project was started in September, 1958. About 25,000 cu yd, therefore, remain to be moved across

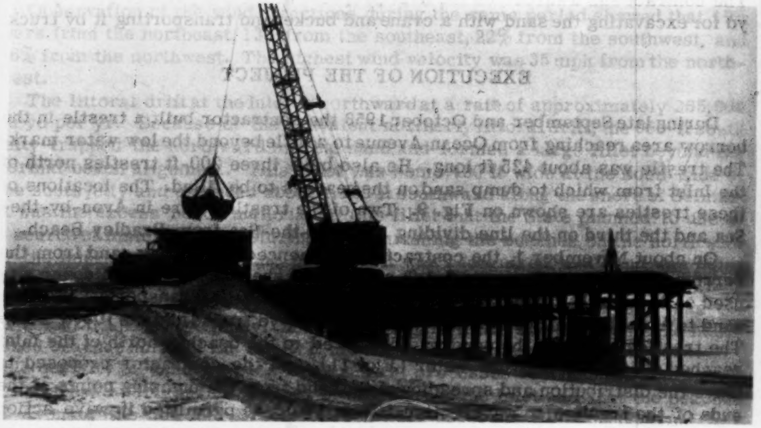

FIG. 10.—EXCAVATING SAND FROM THE BORROW AREA

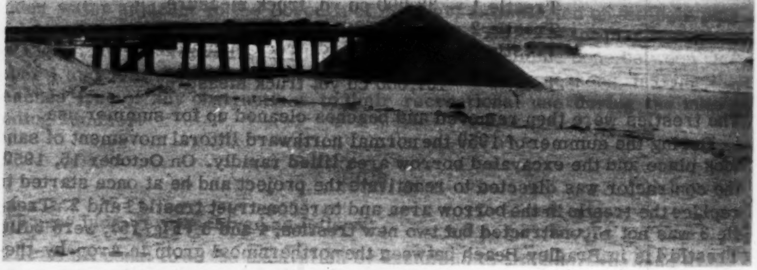

FIG. 11.—SAND DUMPED ON THE BEACH TO BE DISTRIBUTED BY WAVE ACTION AT HIGH WATER

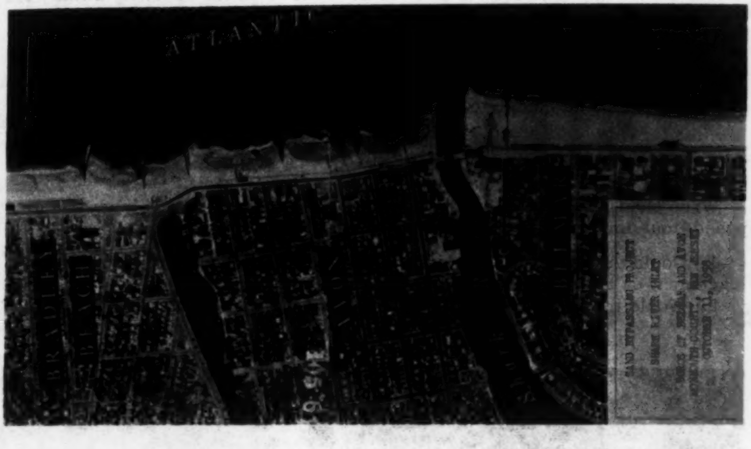
ATLANTI
BRADLEY
BEACH
AVON
BELMAR
SAND BYPASSING PROJECT
SHARK RIVER INLET
BOROUGH OF BELMAR AND AVON
MONMOUTH COUNTY, NEW JERSEY
OCTOBER 11, 1958

FIG. 12.—CONDITION OF THE BEACHES AT THE COMMENCEMENT OF BY-PASSING IN 1958.

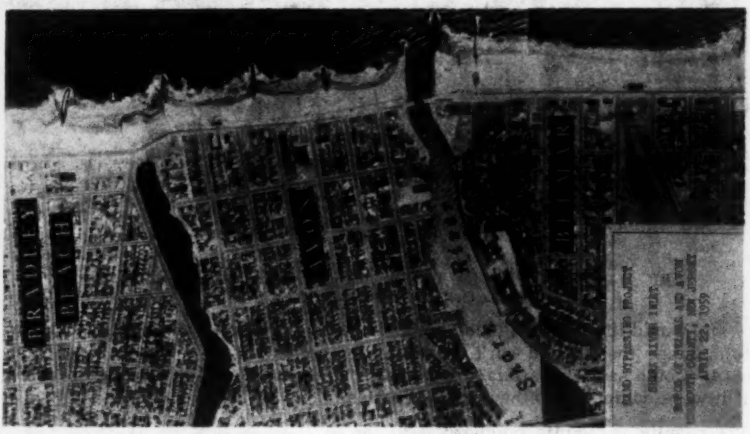
BRADLEY
BEACH
AVON
BELMAR
Shark River
SAND BYPASSING PROJECT
SHARK RIVER INLET
BOROUGH OF BELMAR AND AVON
MONMOUTH COUNTY, NEW JERSEY
APRIL 22, 1959

FIG. 13.—CONDITION OF THE BEACHES AT THE END OF THE FIRST WINTER'S WORK

the inlet. This will undoubtedly be accomplished by early April. Most of the sand is now being delivered over trestles 4 and 5 as the beaches initially fed through trestles 1, 2, and 3 are in excellent condition and did not erode seriously during the past summer (Fig. 13).

It is anticipated that the entire 250,000 cu yd to be moved under the contract will have been by-passed across the Inlet before the middle of April and the project will then be complete.

RESULTS

When sand by-passing by the method used at Shark River Inlet was proposed, four important questions were raised. These were:

1. Could the contractor maintain a satisfactory schedule of excavation and placement by the methods he proposed to use?
2. Would the removal of sand in the planned quantity damage Belmar Beach?

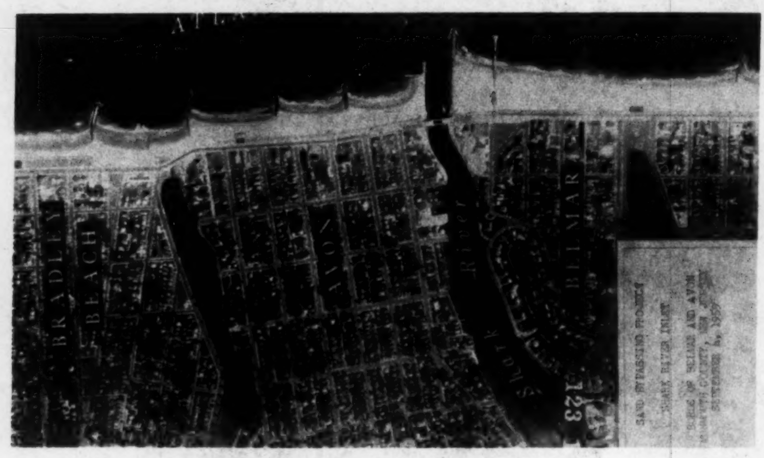

FIG. 14.—CONDITION OF THE BEACHES BEFORE COMMENCEMENT OF THE SECOND WINTER'S WORK

3. Would littoral drift bring material to the borrow area at a rate sufficient to maintain the planned work schedule?
4. Could sand be satisfactorily distributed and spread on the filled beaches if it were dumped onto them from trestles?

The results accomplished show:

a. The contractor has maintained a satisfactory schedule of placement inasmuch as he excavated and placed 137,000 cu yd during the first winter he worked. This is more than half the 250,000 cu yd he was to move during two winters.

b. Aerial photographs and maps (Figs. 12, 13, 14, 15, 16, and 17), show that the alignment of the Belmar beach has been maintained in a very satisfactory way. The portion removed was only that part of the fillet south jetty which was considered excess material.

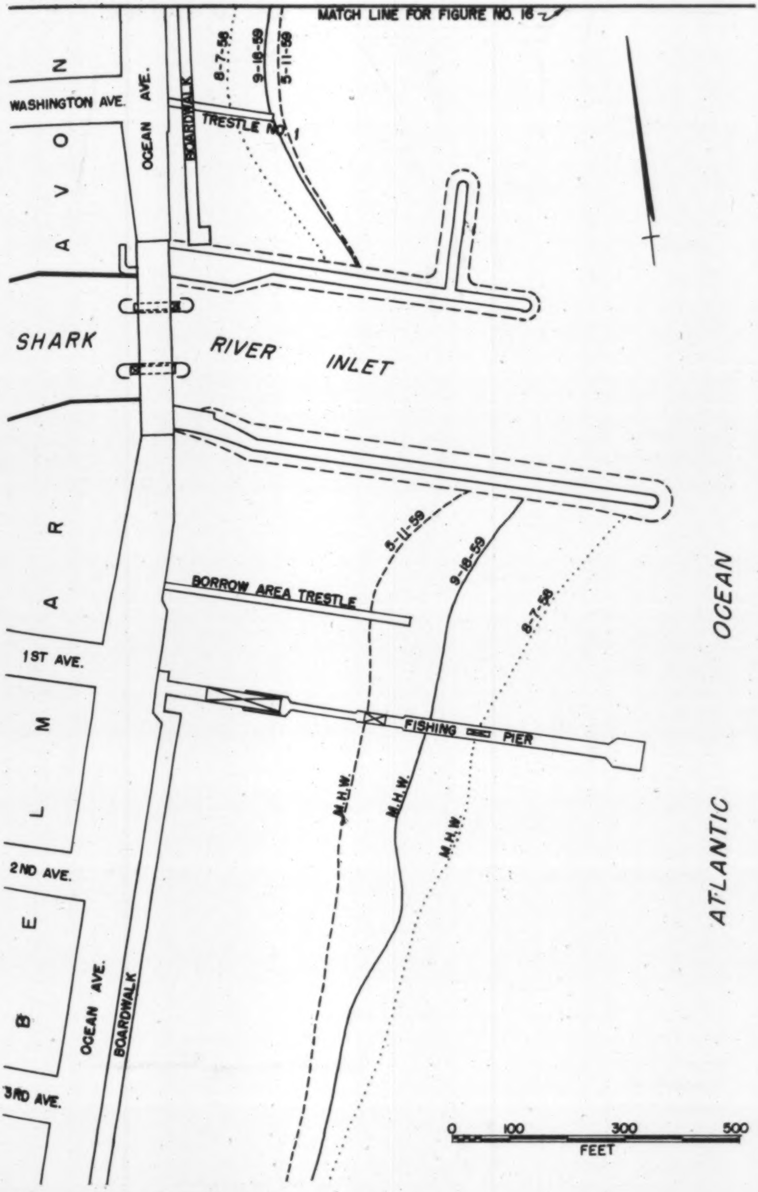
MATCH LINE FOR FIGURE NO. 16
8-7-58
9-18-59
5-11-59
WASHINGTON AVE.
OCEAN AVE.
BOARDWALK
TRESTLE NO. 1
N
A V O N
SHARK RIVER INLET
5-11-59
9-18-59
8-7-58
BORROW AREA TRESTLE
B E L M A R
1ST AVE.
FISHING PIER
M.H.W.
M.H.W.
M.H.W.
2ND AVE.
3RD AVE.
OCEAN AVE.
BOARDWALK
ATLANTIC OCEAN
0
100
300
500
FEET

FIG. 15.—PLAN SHOWING RESULT OF SAND BY-PASSING PROJECT

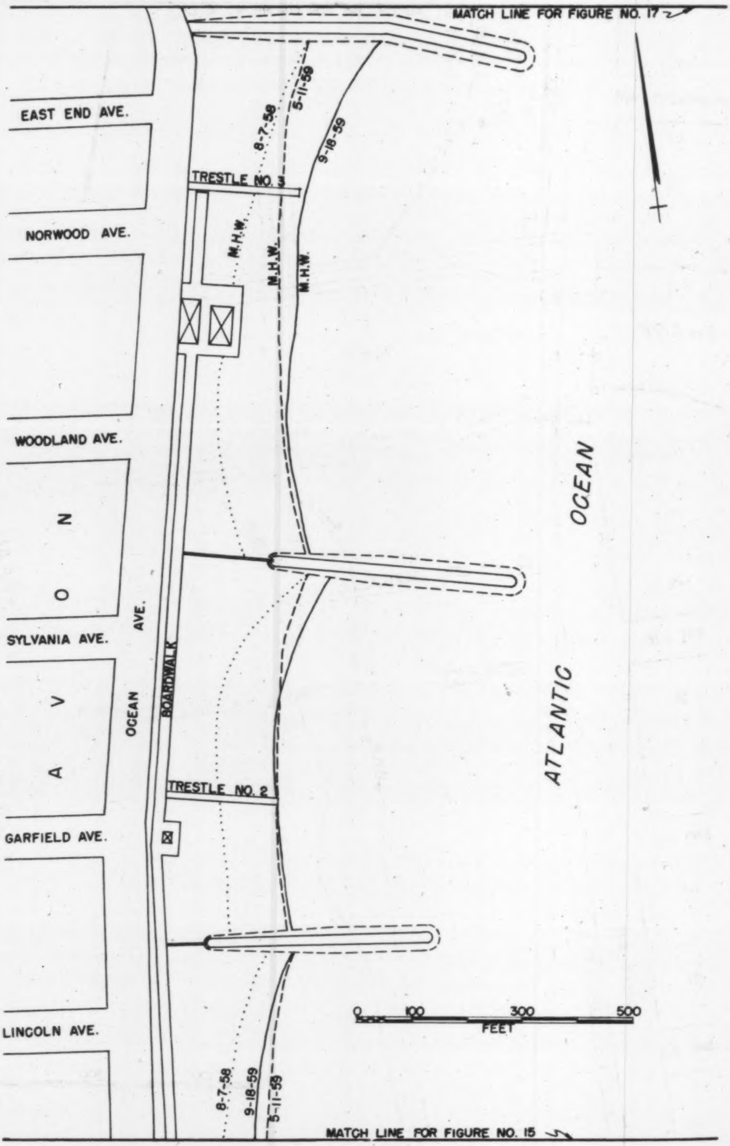
MATCH LINE FOR FIGURE NO. 17
EAST END AVE.
8-7-58
5-11-59
9-18-59
TRESTLE NO. 3
NORWOOD AVE.
M.H.W.
M.H.W.
M.H.W.
WOODLAND AVE.
OCEAN
ATLANTIC
A V O N
SYLVANIA AVE.
OCEAN AVE.
BOARDWALK
TRESTLE NO. 2
GARFIELD AVE.
LINCOLN AVE.
0 100 300 500
FEET
8-7-58
9-18-59
5-11-59
MATCH LINE FOR FIGURE NO. 15

FIG. 16.—PLAN SHOWING RESULT OF SAND BY-PASSING PROJECT

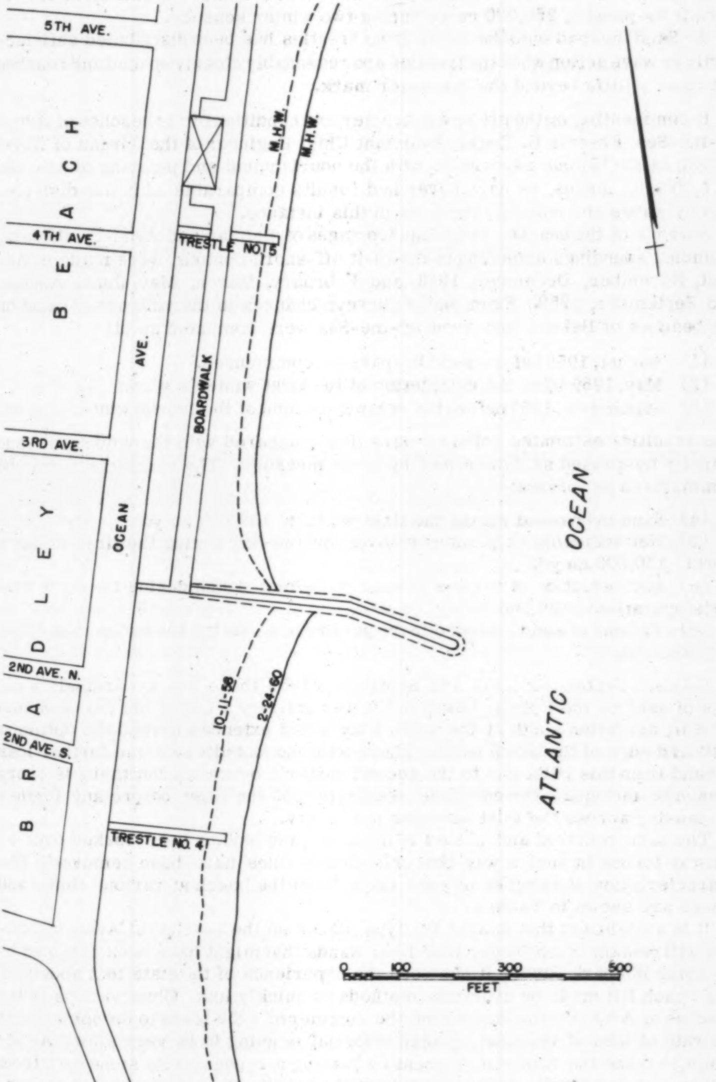
5TH AVE.
4TH AVE.
3RD AVE.
2ND AVE. N.
2ND AVE. S.
B R A D L E Y
B E A C H
OCEAN
AVE.
BOARDWALK
TRESTLE NO. 5
TRESTLE NO. 4
M.H.W.
M.H.W.
10-11-58
2-24-60
ATLANTIC
OCEAN
0
100
300
500
FEET
MATCH LINE FOR FIGURE NO. 16

FIG. 17.—PLAN SHOWING RESULT OF SAND BY-PASSING PROJECT

c. Material has been brought into the borrow area at a sufficient rate to permit by-passing 250,000 cu yd during two winter seasons.

d. Sand dumped onto the beach from trestles has been distributed satisfactorily by wave action when the trestles are reasonably closely spaced and reached out to or a little beyond the low water mark.

In commenting on the present character and condition of the beaches of Avon-by-the-Sea, Francis B. Cogan, Assistant Chief Engineer of the Bureau of Navigation, said: "In our experience with the nourishment and pumping of beaches by hydraulic means, we have never had results comparable with the distribution by waves and natural forces as in this instance."

Surveys of the beaches and ocean frontages of Belmar and Avon-by-the-Sea, including soundings extending to the 25-ft off-shore contour, were made in August, November, December, 1958, and February, March, May, June, August, and September, 1959. From these surveys changes in the volumes of sand on the beaches of Belmar and Avon-by-the-Sea were computed as of:

(1) August, 1958 before sand by-passing commenced.
(2) May, 1959 after the completion of the first winter's work.
(3) September, 1959 before the commencement of the second winter's work.

The resulting estimated volumes were then compared with the volume of sand actually by-passed as determined by truck measure. The comparison may be summarized as follows:

(a) Sand by-passed during the first winter: 137,000 cu yd.

(b) Net accretion on beaches of Avon-by-the-Sea during the first winter's work: 150,000 cu yd.

(c) Net reduction of volume of sand on Belmar beach during the first winter's operation: 129,000 cu yd.

(d) Volume of sand returned to the borrow area during the summer of 1959: 89,000 cu yd.

Between September 1958 and September 1959 there was accordingly a net loss of sand on the Belmar beach but it was entirely a loss from the unwanted fillet or accretion south of the south jetty which extended beyond the optimum outboard edge of the beach berm. There will undoubtedly be some further loss of sand from this fillet due to the second winter's operation but there is every reason to anticipate the complete restoration of the fillet before any further by-passing across the Inlet becomes necessary.

The sand removed and placed is medium sand having been worked over by littoral forces in such a way that undesirable fines have been removed. The characteristics of samples of sand taken from the beach at various times and places are shown in Table 1.

It is anticipated that sand of the type placed on the beaches of Avon-by-the-Sea will remain much longer than finer sands that might have been obtained by dredging in Shark River. It has been the experience of the State that about 25% of a beach fill made by hydraulic methods is quickly lost. Observations of the beaches of Avon-by-the-Sea during the summer of 1959 leads to an opinion that the rate of loss of the newly placed material is going to be very slow. An attempt to trace the rate of movement by placing a recognizable stone dust from a quarry at Somerville is being tried but the results are not considered encouraging.

TABLE 1.—SAND CHARACTERISTICS

Sample (1)	Date (2)	Median Diameter (3)	Coefficient of sorting (4)	Skewness (5)
Avon Trestle No. 2	Sept. 1958	.43	1.41	1.08
Avon Trestle No. 2	Oct. 1959	.72	1.69	0.97
Avon Trestle No. 1	Oct. 1959	.62	1.49	0.97
Avon Trestle No. 3	Oct. 1959	.51	1.67	1.17
Bradley Beach Trestle No. 4	Oct. 1959	.37	1.21	1.04
Belmar - Borrow Area	Jan. 1959	.98	1.53	1.00
Belmar - Borrow Area	Oct. 1959	1.24	1.46	0.96
Belmar - 2nd Avenue	Oct. 1959	1.20	1.39	0.92
Belmar - 4th Avenue	Oct. 1959	0.56	1.23	1.03
Belmar - 6th Avenue	Oct. 1959	0.50	1.27	0.89

ACKNOWLEDGMENTS

The success of the project must be attributed largely to the care and attention to its direction given personally by Thomas Procter, the contractor, and F. B. Cogan.

The author wishes to acknowledge his indebtedness to those who gave the encouragement, advice, and assistance which made the preparation of this paper possible. Among those who were most helpful should be mentioned Commissioner Salvatore A. Bontempo, of the Department of Conservation and Economic Development of the State of New Jersey; Director Kenneth Kreveling, of the Department's Planning Division; Peter Gannon, Chief of the Bureau of Navigation; James K. Rankin, M. ASCE, the Bureau's Chief Engineer; and Francis B. Cogan, Assistant Chief Engineer of the Bureau. Finally, Harold Shinn must be thanked for preparing the drawings used as figures.

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TRANSACTIONS

Paper No. 3172

PNEUMATIC BARRIER AGAINST SALT WATER INTRUSION

By Ian Larsen[1]

SYNOPSIS

In rivers subject to salt water intrusion, bubbles of air from perforated pipes on the river bottom will create an upward flow of salt water and thus a mixing between the fresh and salt water layers. When the upward flow reaches the absolute maximum of salt water discharge the salt water will be unable to penetrate the bubble "curtains." Such installations may provide the answer to the salt water problem in many rivers.

INTRODUCTION

The difference in density is known to be the cause of salt water intrusion in estuaries. In this connexion, three types of intrusion are recognized:

1. Estuaries with a salt water wedge, that is, the salt water, sharply separated from the fresh water, intrudes along the river bed in the shape of a wedge.
2. Well mixed estuaries, which are generally characterized by vertical isohalines.
3. Partly mixed estuaries, that form the transition between the two types already mentioned.

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The mixing conditions in an estuary seem to be determined chiefly by the ratio between the fresh water discharge in a tidal period and the tidal prism.[2]

In many places in the world salt water pollution creates serious problems with regard to both agriculture and water supplies. Therefore, it is valuable to develop methods of hindering this intrusion.

Training walls, sills, and other control structures have been used in the fight against the salt water, but their effectiveness is limited because of navigational requirements. It is believed that the pneumatic salt water barrier, either alone, or in combination with the previously mentioned structures, may become a valuable means for the control of salt water intrusion.

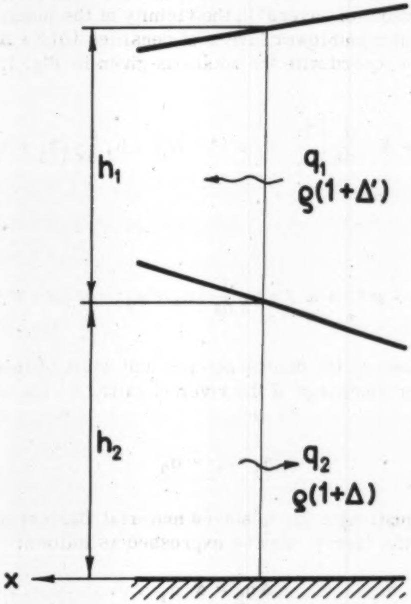

FIG. 1.—NOTATIONS OF TWO-LAYER FLOW AT RIVER MOUTH

The fundamental principle of the pneumatic salt water barrier is that the bubbles of air emitted from perforated pipes placed on the river bed will create an upward flow of salt water that will be mixed with the upper layer of outgoing fresh water. In partly mixed estuaries and estuaries with a salt water wedge, the natural velocities in the lower layer of salt water are small. Application of a pneumatic barrier will change the hydraulic conditions of the barrier downstream and at the river mouth especially. Therefore, as an introduction to the description of the pneumatic barrier it is necessary to consider the hydraulic conditions at the river mouth.

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[2] "Some Effects of Upland Discharge on Estuarine Hydraulics," by Henry B. Simmons, Proc. Paper 792, ASCE, Vol. 81, September, 1955.

GENERAL THEORY OF TWO-LAYER DENSITY FLOW AT RIVER MOUTHS

At a river mouth or at any other location where an abrupt widening of the cross-section occurs, there is a sudden decrease in the thickness of the upper layer.

For the stability conditions of this hydraulic jump long, internal waves are stationary. Neglecting the shear stresses at the bottom and at the interface, as well as the mixture of layers in the vicinity of the jump, the equilibrium equations for the upper and lower layers of densities ($\rho(1 + \Delta')$ and $\rho(1 + \Delta)$, respectively) are in accord with the notations given in Fig. 1:

$$\rho\ (1 + \Delta')\ \frac{d}{dx}\left(\frac{q_1^2}{h_1}\right) = -\rho\ (1 + \Delta')\ g\ h_1\ \frac{d}{dx}\left(h_1 + h_2\right) \quad \ldots\ldots \quad (1)$$

and

$$\rho\ (1 + \Delta)\ \frac{d}{dx}\left(\frac{q_2^2}{h_2}\right) = -\ \rho\ (1 + \Delta')\ g\ h_2\ \frac{d}{dx}\left(h_1 + h_2\right) - \rho\ (\Delta - \Delta')\ g\ h_2\ \frac{dh_2}{dx} \quad \ldots \quad (2)$$

in which q_1 and q_2 denote the discharges per unit width of flow.

If the fresh water discharge of the river is called q_o, the continuity equation of the water gives:

$$q_1 = q_o + q_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

The continuity equation for the dissolved material, that creates the difference in density between the layers, may be expressed as follows:

$$\Delta\, q_2 = \Delta'\ q_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

If ψ denotes the ratio between the discharge of q_2 of the lower layer and the fresh water discharge q_o, Eq. 3 can be converted to:

$$q_1 = (1 + \psi)\ q_o \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

which when inserted in Eq. 4, gives:

$$\frac{\Delta'}{\Delta} = \frac{\psi}{1 + \psi} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

Eqs. 1 and 2 can be made dimensionless by means of the parameters:

$$\xi = \frac{x}{h_o} \qquad (7)$$

$$\eta_1 = \frac{h_1}{h_o} \qquad (8)$$

$$\eta_2 = \frac{h_2}{h_o} \qquad (9)$$

and

$$F^{-2} = \frac{g\,h_o^3}{q_o^2} \qquad (10)$$

in which h_o is the depth of the channel and F is the Froude number of the river. Thus, Eqs. 1 and 2 become

$$(1 + \Delta')\frac{d}{d\xi}\left[\frac{(1+\psi)^2}{\eta_1}\right] = -(1+\Delta')\,F^{-2}\,\eta_1\frac{d}{d\xi}\left(\eta_1 + \eta_2\right) \qquad (11)$$

and

$$(1 + \Delta)\frac{d}{d\xi}\left(\frac{\psi^2}{\eta_2}\right) = -(1+\Delta')\,F^{-2}\,\eta_2\frac{d}{d\xi}\left(\eta_1 + \eta_2\right) - (\Delta - \Delta')\,F^{-2}\,\eta_2\frac{d}{d\xi}\,\eta_2 \qquad (12)$$

By differentiation of the individual terms and regrouping, the following is obtained:

$$\left[\frac{(1+\psi)^2}{\eta_1^2} - \eta_1\,F^{-2}\right]\frac{d\eta_1}{d\xi} = \eta_1\,F^{-2}\frac{d\eta_2}{d\xi} \qquad (13)$$

and

$$(1 + \Delta')\,\eta_2\,F^{-2}\frac{d\eta_1}{d\xi} = (1+\Delta)\left(\frac{\psi^2}{\eta_2^2} - \eta_2\,F^{-2}\right)\frac{d\,\eta_2}{d\,\xi} \qquad (14)$$

The previously mentioned stability condition of the hydraulic jump involves Eqs. 13 and 14 being satisfied by any value of $\frac{d\eta_1}{d\,\xi}$ and $\frac{d\eta_2}{d\,\xi}$. In other words, the equations must be identical. Hence

$$(1+\Delta')\,\eta_2\,\eta_1\,F^{-2} = (1+\Delta)\left(\frac{\psi^2}{\eta_2^2} - \eta_2\,F^{-2}\right)\left[\frac{(1+\psi)^2}{\eta_1^2}\,F^2 - \eta_1\right] \;..(15)$$

or

$$\frac{\Delta-\Delta'}{1+\Delta}\,\eta_1\,\eta_2\,F^{-2} = \frac{\eta_1}{\eta_2^2}\psi^2 + \frac{\eta_2}{\eta_1^2}(1+\psi)^2 - \frac{\psi^2\,(1+\psi)^2}{\eta_2^2\,\eta_1^2}\,F^2 \;...(15a)$$

Inserting Δ' from Eq. 6 results in

$$\frac{\Delta}{1+\Delta}\;\frac{1}{1+\psi}\;\eta_1^3\,\eta_2^3\,F^{-2} = \psi^2\,\eta_1^3 + (1+\psi)^2\,\eta_2^3 - \psi^2\,(1+\psi)^2\,F^2 \;..(15b)$$

In Eq. 15b the usual densimetric Froude number F_Δ as defined by Eq. 16 may be introduced

$$F_\Delta^{-2} = \frac{\Delta}{1+\Delta}\;F^{-2} \;\ldots\ldots\ldots\ldots\ldots\ldots(16)$$

Since F^2 is normally small and

$$\eta_1 + \eta_2 \sim 1 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots(17)$$

Eq. 15b may be approximated by:

$$\phi\;(\eta_1\,\psi) = \psi^2\,\eta_1^3 + (1+\psi)^2\,(1-\eta_1)^3 - \frac{1}{1+\psi}\,\eta_1^3\,(1-\eta_1)^3\,F_\Delta^{-2} = 0 \;..(18)$$

Now, η_1 lays in the interval between 0 and 1. Further analysis of Eq. 18 shows that for each value of F_Δ there exists a certain value of ψ, ψ_m, so that for $0 \leq \psi < \psi_m$ Eq. 18 has two solutions for η_1, the larger solution corresponding to an unstable condition. Values of $\psi > \psi_m$ will result in Eq. 18 having no roots. For $\psi = \psi_m$, Eq. 18 has a double root, η_m. For $\psi = 0$, Eq. 18 has, apart from the unusable root $\eta_1 = 1$, the root:

$$\eta_1 = \eta_x = F_\Delta^{2/3} \;\ldots\ldots\ldots\ldots\ldots\ldots(19)$$

In the case of a stationary salt water wedge, in which the natural mixing between the layers causes only low velocities in the salt water layer, η_x denotes the dimensionless thickness of the layer of fresh water at the river mouth. Fig. 2 shows the roots of Eq. 18 for various values of η_x, while the correlation between η_x and the densimetric Froude number F_Δ is given in Fig. 3.

Eq. 18 yields the double root, η_m, when

$$\left(\frac{\delta\phi}{\delta\eta}\right)_{\eta = \eta_m} = 3 (\psi_m^2 \eta_m^2 - \left(1 + \psi_m\right)^2 \left(1 - \eta_m\right)^2 - \frac{1}{1 + \psi_m} \eta_m^2 \left(1 - \eta_m\right)^2$$

$$\left(1 - 2\,\eta_m\right) F_\Delta^{-2} = o \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

In this case Eq. 18 gives:

$$\psi_m^2 \eta_m^3 + \left(1 + \psi_m\right)^2 \left(1 - \eta_m\right)^3 - \frac{1}{1 + \psi_m} \eta_m^3 \left(1 - \eta_m\right)^3 F_\Delta^{-2} = o \ldots (21)$$

Eliminating F_Δ from Eqs. 20 and 21 we find:

$$\eta_m = \frac{1}{1 + \sqrt{\frac{\psi_m}{1 + \psi_m}}} \ldots\ldots\ldots\ldots\ldots (22)$$

The relation between ψ_m and F_Δ may be found by inserting Eq. 22 into Eq. 18, which yields

$$F_\Delta^{-2} = \left(1 + \psi_m\right) \left(\sqrt{1 + \psi_m} + \sqrt{\psi_m}\right)^4 \ldots\ldots\ldots\ldots (23)$$

Fig. 3 shows ψ_m, η_m and η_K versus F_Δ^{-2}.

The quantity of salt water necessary to create the double-root condition can be expressed by ζ, and defined by

$$\zeta = \frac{q_{2,m}}{\sqrt{\Delta g h_o^3}} = \frac{\psi_m q_o}{\sqrt{\Delta g h_o^3}} \sim \psi_m F_\Delta = \frac{\psi_m}{\sqrt{1 + \psi_m}\left(\sqrt{1 + \psi_m} + \sqrt{\psi_m}\right)^2} \ldots (24)$$

In Fig. 3 ζ is shown as a function of F_Δ^{-2}. The term ζ has a maximum value of

$$\zeta_{max} = 0.122 \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

that corresponds to

$$q_{2,max} = 0.122 \sqrt{\Delta g h_o^3} \ldots\ldots\ldots\ldots\ldots (26)$$

BASIC PRINCIPLE OF THE SALT WATER BARRIER

The stationary salt water wedge has been previously considered, in which the flow in the salt water layer was moderate in relation to the flow in the upper layer. The dimensionless thickness of the upper layer at the river mouth was given by Eq. 19.

Now assume that the vertical mixing between the two layers is intensified. The upper layer becomes increasingly saline, the inward and outward currents are increased simultaneously, and the thicknesses of the two layers of the river mouth can be determined by Eq. 18.

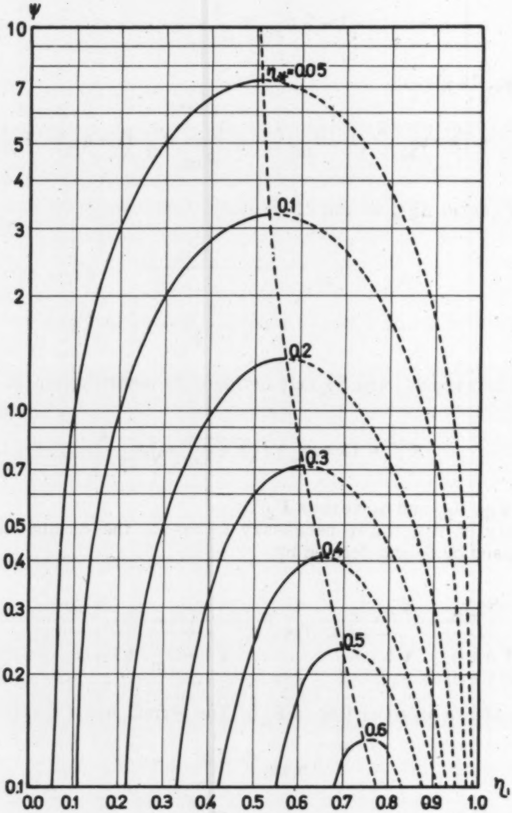

FIG. 2.—THE DIMENSIONLESS THICKNESS OF THE FRESH WATER LAYER η_1 AT THE RIVER MOUTH

However, it is only possible by this means to increase the flows in the upper and lower layers to a limit corresponding to ψ_m in Eq. 23. Further attempts to intensify the mixing will have no result. This fact has been proved in experiments by Stommel and Farmer.[3]

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[3] "Control of Salinity in an Estuary by a Transition," by Henry Stommel and Harlow G. Farmer, Journal Mar. Res., Vol. XII, 1953, pp. 13-20.

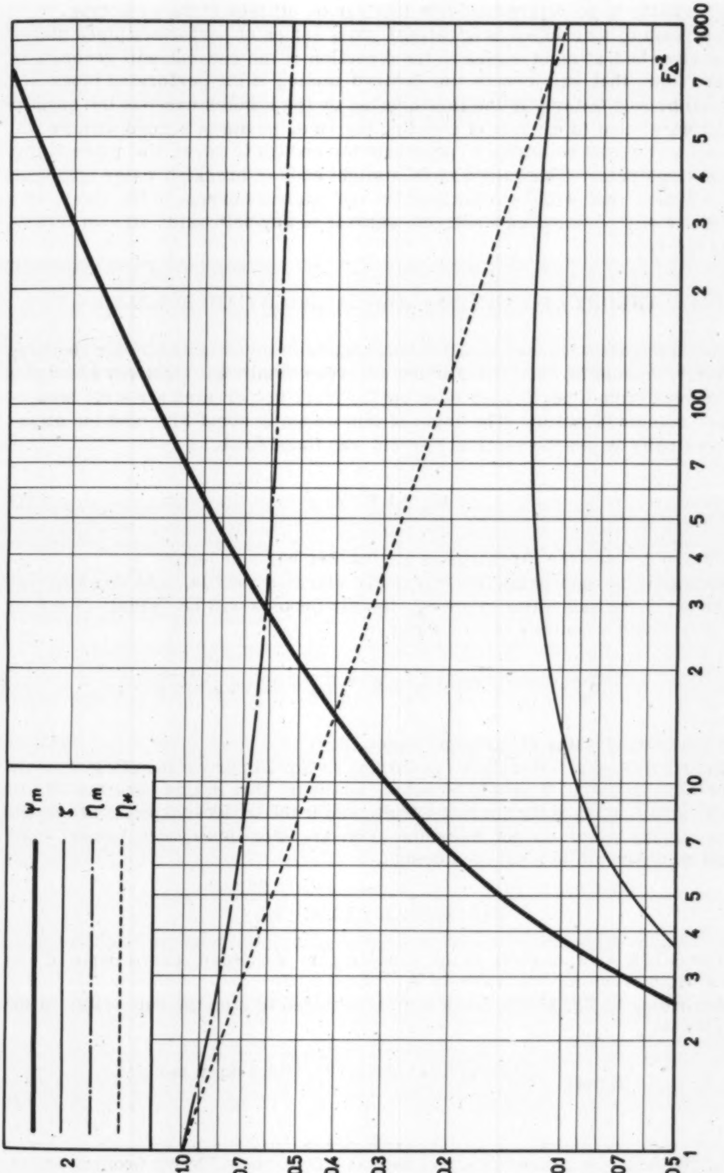

FIG. 3.—ψ_m, η_m, η_* AND ζ VERSUS F_Δ^{-2}

It remains to be determined how this can be utilized in the prototype.

A stream of air bubbles originating from a series of perforated pipes placed on the river bed, at right angles to the direction of the current, will generate an upward flow, that will create the desired mixing if the perforated pipes are covered by salt water. If the flow created by the bubbles exceeds the quantity of salt water that is capable of invading the river mouth in accord with Eq. 26, the salt water will be unable to penetrate far enough to cover the pipes laying furthest upstream. Thus, apart from a slight eddy of brackish water upstream of the tubes, that will be weakened by the natural current in the river, this equipment will form an impenetrable barrier to any salt water stream threatening the river.

THEORY OF UPWARD FLOW CAUSED BY AIR BUBBLES

The upward flow caused by air bubbles has been investigated by Sir Geoffrey Taylor.[4] Assuming that the bubbles are very small, Sir Geoffrey found that air bubbles from a perforated tube on the river bed created a rising, wedge-shaped column of water. The angle of this wedge is about 31°, and the maximum velocity W_o in the vertical current was found to be

$$W_o = 1.9\ \sqrt[3]{v\,g} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (27)$$

in which v denotes the discharge of air bubbles per unit length.

Assuming an approximately parabolic velocity profile, and denoting the thickness of the salt water layer h_s, the air bubbles will be capable of mixing per unit length a quantity of salt water

$$q_s = \frac{2}{3}(2\ t\ g\ 15^\circ)\ 5\ h_s\ 1.9\ \sqrt[3]{v\,g} = 0.71\ h_s\ \sqrt[3]{v\,g} \quad \ldots\ldots \quad (28)$$

with the flowing water of the upper layer.

Should this salt water flow, according to Eq. 26, prove insufficient, more tubes can be used. It should be noted, however, that in this case Eq. 28 will hardly give a completely correct result even if the wedge-shaped zones of upward-moving water do not intermix (Fig. 4). For this reason, model tests would be desirable in a detailed design.

NUMERICAL EXAMPLES

The GÖTA ÄLV, a river in the western part of Sweden, has a depth of 6 m and a width of 50 m. The value of Δ is about 0.02.

According to Eq. 26 the vertical flow required to stop the salt water at the barrier is:

$$q_{2,max} = 0.122\ \sqrt{(0.02)(9.81)(6)^3} = 0.8 \text{ sq m per sec}$$

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[4] "The Action of a Surface Current Used as a Breakwater," by Sir Geoffrey Taylor, Proceedings, Royal Soc., London, Series A, Vol. 231, 1955, p. 466.

If it is estimated that this upward flow must be fully developed $\frac{1}{10}$ h_o over the river bed and if 8 pipes are used, the air consumption per unit width is found from

$$q_{2,max} \doteq 0.8 = 8\ (0.71)(0.1)(6)\ \sqrt[3]{v\ 9.81}$$

whence

$$v = 0.0013 \text{ sq m per sec per pipe}$$

The total air consumption is

$$V_{total} = 8(0.0013)\ 50 = 0.52 \text{ cu m per sec}$$

It may be mentioned that exceptionally low discharges of 50 and 25 cu m per sec give values for F_{Δ}^{-2} of 40 and 160, respectively. The south and south-west passes of the Mississippi River are 11 m deep and the sum of their widths is 750 m. The value of Δ is taken as 0.02. It is found that

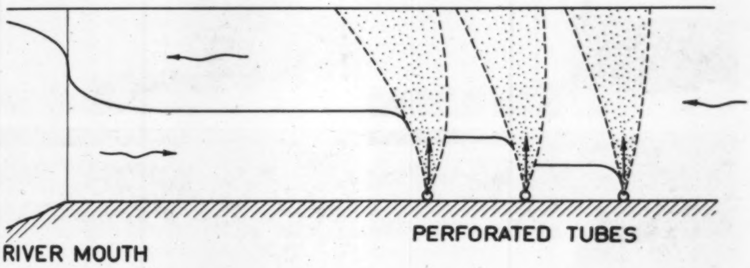

FIG. 4.—PNEUMATIC SALT WATER BARRIER

$$q_{2,max} = 0.122\ \sqrt{(0.02)(9.81)(11)^3} = 2 \text{ sq m per sec}$$

When 10 pipes are used, the air consumption is found in the same manner:

$$q_{2,max} = 2.0 = 10(0.71)(0.1)(11)\ \sqrt[3]{v\ 9.81}$$

$$v = 0.0017 \text{ sq m per sec per pipe}$$

and

$$V_{total} = 10\ (0.0017)\ 750 = 12{,}7 \text{ cu m per sec}$$

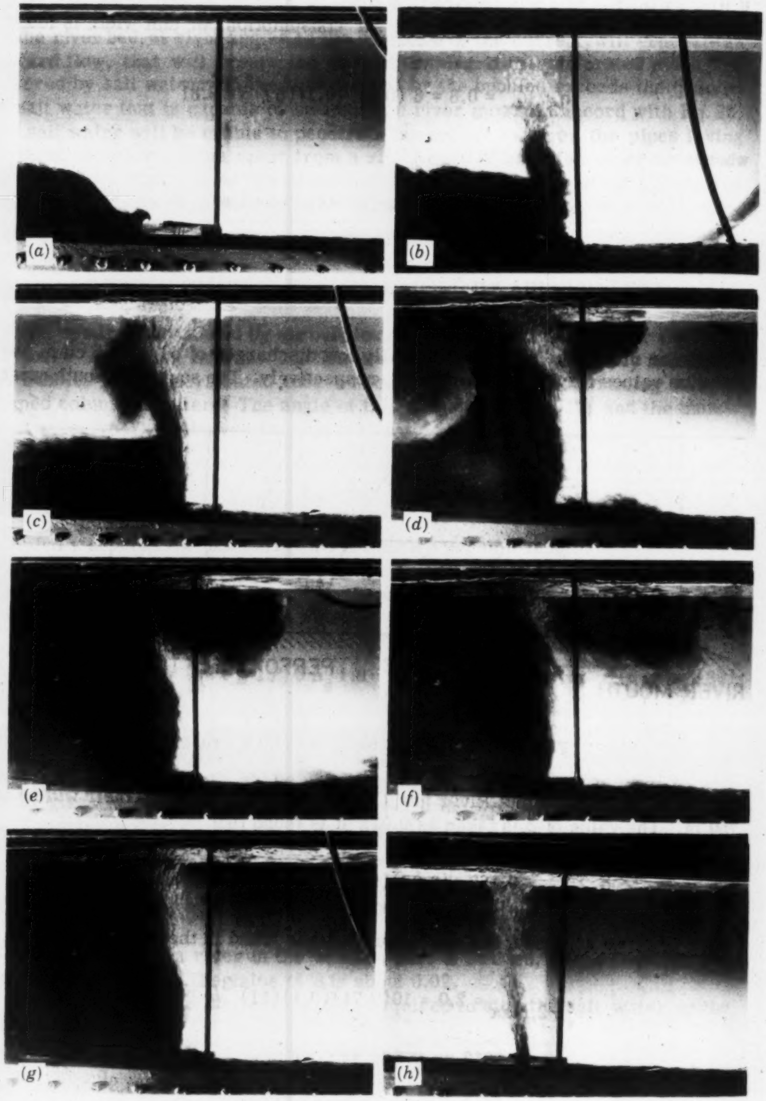

FIG. 5.—MODEL TESTS WITH $\mathsf{F}_{\Delta}^{-2} = 125$

In the Pass A Loutre in the Mississippi River, h_o is 8 m and the width 1000 m. Thus:

$$q_{2,max} = 1.2 \text{ sq m per sec}$$

and corresponding to 10 pipes

$$V_{total} = 10 \text{ cu m per sec}$$

A total discharge in the Mississippi River of 2000 cu m per sec gives values for F_{Δ}^{-2} of about 125 in the three passes.

MODEL TESTS

Model tests have been made with values for F_{Δ}^{-2} of between 100 and 200. Fig. 5 shows a series of photos from a run with $F_{\Delta}^{-2} = 125$, corresponding to $q_o = 64.0$ sq cm per sec, $h_o = 23.5$ cm and $\Delta = 0.04$. In these tests, the salt water was represented by means of a dyed sugar solution.

CONCLUSIONS

Pneumatic salt water barriers of the magnitudes found in the preceding numerical examples may be established at costs considerably lower than the construction costs of the usual, less effective, salt water control structures. The running costs of the pneumatic barriers are difficult to estimate in general terms because these costs depend on the variations in the river discharge.

The upward-moving stream of air bubbles will not affect the river traffic, but, the radical change in the current conditions at the river bed will alter the natural sedimentation conditions in the river. Both bed load transport and a considerable part of the suspended load will tend to deposit in the immediate neighbourhood of the barrier.

As the pneumatic salt water barrier is only intended to operate when the discharge in the river is incapable of creating satisfactory fresh water conditions. That is, during periods of greatly diminished material transport, the possible difficulties caused by the barrier with regard to sedimentation will probably be slight.

It was assumed in the introduction that there is an outward surface current. This is not the case at the mouth of well mixed estuaries. Therefore, it is not possible in this way to improve the conditions downstream of the cross-section where the surface current at flood tide becomes zero. However, upstream of this point, the effect will be the same as previously stated. It should be stated that at irregularly shaped river mouthes or at river mouthes where tides play a dominating role, model tests with the pneumatic salt water barrier will be indispensable.

ACKNOWLEDGMENT

The writer wishes to express his appreciation to Fechn. H. Lundgren, head of the Coastal Engineering Laboratory of the Technical University of Denmark, under whose supervision the work was carried out.

ADDITIONAL REFERENCE

"Abrupt Change in Width in Width in Two-Layer Open Channel Flow," by Henry Stommel and Harlow G. Farmer, Journal Mar. Res. Vol. XI, No. 2, 1952, p. 205.

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TRANSACTIONS

Paper No. 3173

STUDIES OF A CHANNEL THROUGH PADRE ISLAND, TEXAS

By E. A. Hansen,[1] M. ASCE

SYNOPSIS

The problems of tidal hydraulics encountered in design of an artificial inlet from a gulf to an almost tideless bay are numerous. This paper presents the results of several years' observation of tide and current phenomena in such an inlet, and the studies made to determine the best design for a jettied navigation channel through the inlet.

INTRODUCTION

A recent project of considerable interest from the standpoint of tidal hydraulics and beach erosion is the Port Mansfield Channel through Padre Island on the lower coast of Texas. This channel was dredged by the Willacy County Navigation District to provide a navigation route between the coastal bay and the Gulf of Mexico. Two jetties were constructed on the Gulf end to afford protection to the entrance. The construction of this artificial pass afforded opportunity to observe the effects of the disturbance of the natural equilibrium on the Gulf shore by the construction of the jetties and channel. The data and information presented herein were collected by observations extending over a period of approximately 2 yr, following the completion of the Port Mansfield Channel,

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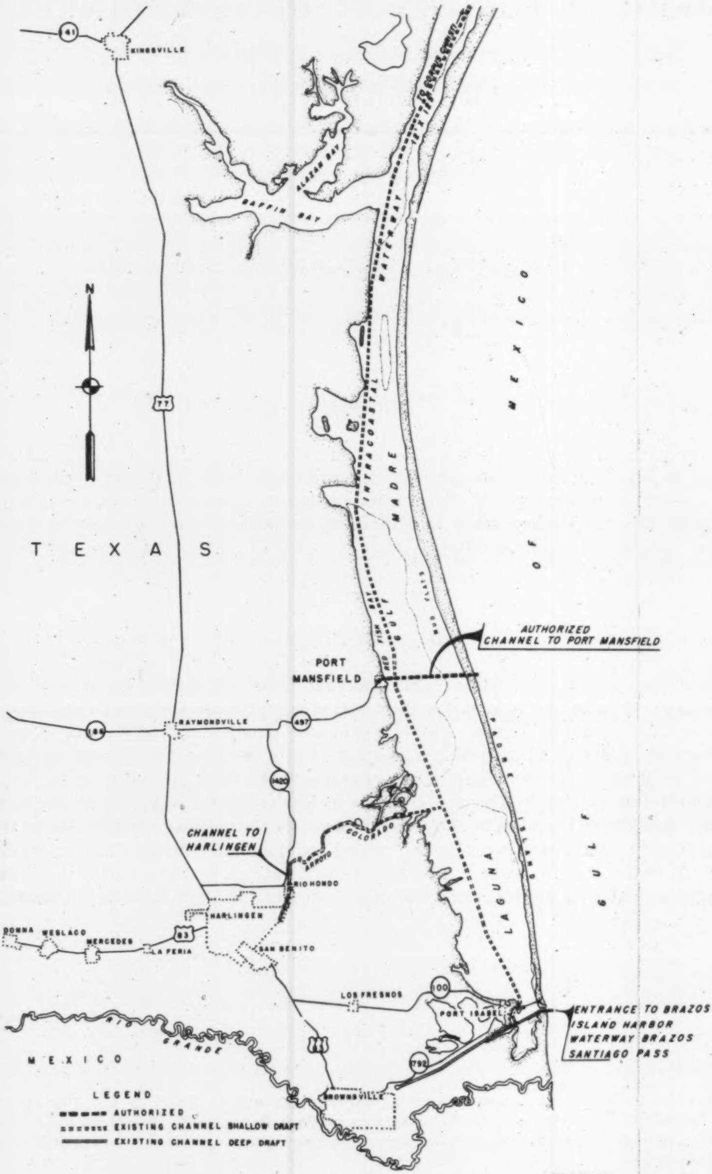
141
KINGSVILLE
BAFFIN BAY
ALAZAN BAY
INTRACOASTAL WATERWAY
N
77
T E X A S
MADRE
GULF
FLATS
MUD
RED FISH BAY
MEXICO
OF
GULF
AUTHORIZED
CHANNEL TO PORT MANSFIELD
PORT
MANSFIELD
186
RAYMONDVILLE
497
1420
CHANNEL TO
HARLINGEN
ARROYO
COLORADO
RIO HONDO
HARLINGEN
DONNA
WESLACO
MERCEDES
83
LA FERIA
SAN BENITO
LAGUNA
ISLAND
PADRE
LOS FRESNOS
100
PORT ISABEL
ENTRANCE TO BRAZOS
ISLAND HARBOR
WATERWAY BRAZOS
SANTIAGO PASS
RIO GRANDE
MEXICO
77
792
BROWNSVILLE
LEGEND
AUTHORIZED
EXISTING CHANNEL SHALLOW DRAFT
EXISTING CHANNEL DEEP DRAFT
SCALE OF MILES

FIG. 1.—PORT MANSFIELD PROJECT LOCATION

and information on studies and conclusions regarding reconstruction of the entrance jetties and channel. The Port Mansfield Channel and vicinity are shown in Fig. 1.

DESCRIPTION

The Port Mansfield Channel lies on the south coast of Texas approximately 93 mi south of Corpus Christi and 38 mi north of Port Isabel. It crosses the Laguna Madre, a long shallow body of water extending along the Texas coast from Corpus Christi on the north to the Brazos Santiago Pass on the south. Laguna Madre, approximately 150 mi long and from 2 to 9 mi wide is separated from the Gulf of Mexico by Padre Island, a virtually uninhabited, low-lying, offshore bar formation. Padre Island varies from 1/2 to 2 mi in width with surface elevations ranging from 2 ft to 16 ft above mean sea level. Higher elevations are all located on a narrow strip of large active sand dunes along the Gulf-beach.

The Laguna Madre is generally divided into two natural bays, separated in the middle by an area of mud flats. The lower bay is approximately 55 mi long and is again divided by a series of shallow flats off the mouth of the Arroyo Colorado which lies approximately 10 mi below Port Mansfield. The bay into which the Port Mansfield Channel enters is locally known as Red Fish Bay and is 28 mi long and 2 to 5 mi wide. Natural depths vary up to a maximum of 8 ft. The ordinary range of tides in the lower Laguna opposite Brazos Santiago Pass is 1-1/2 ft. Tidal effects decrease north of the pass and are barely noticeable in the vicinity of Port Mansfield. Winds cause considerable fluctuations in the water surface elevation. At Brazos Santiago Pass strong north winds may raise the water surface as much as 3 ft above normal tide.

The authorized Federal project for the Gulf Intracoastal Waterway extends through the center of Laguna Madre along its entire length from Corpus Christi Bay to Brazos Santiago. The main channel of the Intracoastal Canal is 12 ft deep and 125 ft wide on the bottom. A branch channel has been extended westward from the main channel in Red Fish Bay to a turning basin at the town of Port Mansfield.

EXISTING IMPROVEMENTS

In September 1957, the Willacy County Navigation District completed construction of additional improvements to Port Mansfield. The work performed included additional harbor basins, a shallow-draft outlet channel through Laguna Madre and Padre Island to the Gulf of Mexico, and jetties at the Gulf entrance. These improvements are shown in Fig. 2.

The new outlet channel to the Gulf of Mexico extends eastward from the junction of the authorized Federal project channel to Port Mansfield and the main channel of the Gulf Intracoastal Waterway, across Laguna Madre and Padre Island to the Gulf. The channel, from the Federal project to the easterly side of Padre Island, was dredged 10 ft deep and 100 ft wide. From Padre Island through the jetties to the 16-ft depth in the Gulf, the channel was dredged 16 ft deep and 250 ft wide. The new channel from the Gulf Intracoastal Waterway to the Gulf is 8.6 miles in length, and the overall length of channel from the Gulf to Port Mansfield, including the authorized tributary channel and turning basin, is 10.0 miles.

TETRAPOD JETTIES

The two jetties constructed by the local interests were parallel to, and 500 ft distant from the channel centerline and extended from the east shore of Padre

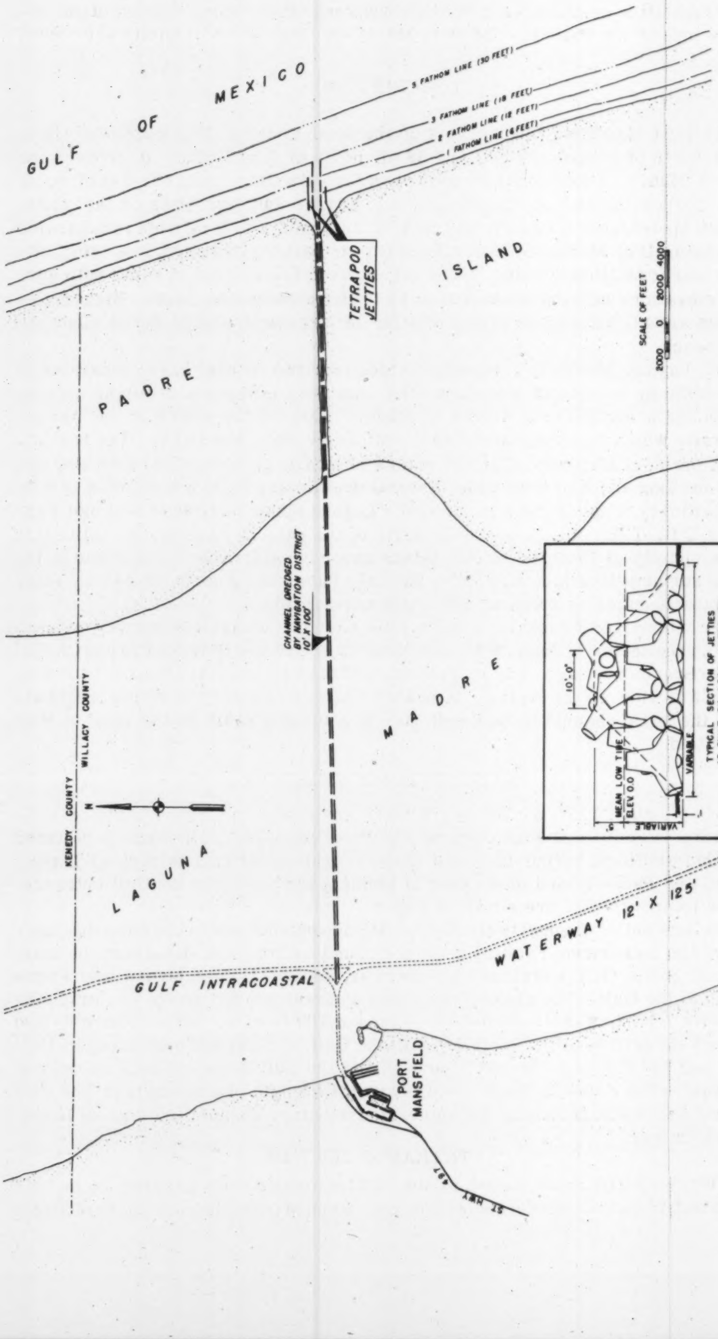
GULF OF MEXICO
5 FATHOM LINE (30 FEET)
3 FATHOM LINE (18 FEET)
2 FATHOM LINE (12 FEET)
1 FATHOM LINE (6 FEET)
PADRE ISLAND
TETRAPOD JETTIES
SCALE OF FEET
2000 0 2000 4000
CHANNEL DREDGED BY NAVIGATION DISTRICT 10' X 100'
LAGUNA MADRE
KENEDY COUNTY
WILLACY COUNTY
N
GULF INTRACOASTAL WATERWAY 12' X 125'
PORT MANSFIELD
ST HWY 497
10'-0"
MEAN LOW TIDE ELEV 0.0
5'
VARIABLE
VARIABLE
TYPICAL SECTION OF JETTIES AS BUILT BY LOCAL INTERESTS

FIG. 2.—LOCAL INTERESTS IMPROVEMENTS, PORT MANSFIELD, TEXAS

Island into the Gulf of Mexico. The north jetty was 1,600 ft long and extended to the 15-ft depth contour. The south jetty was 900 ft long and extended to the 10-ft contour. The crown elevation of both jetties was generally 5 ft above mean low tide and the top width was 10 ft. The jetties were formed by placing precast concrete pieces of geometric design. These pieces, known as tetrapods, were placed on the sand, without a stone blanket to distribute the load or prevent erosion. The tetrapods consist of four symmetrically placed truncated conical legs emanating from a common center. The unit weights of the blocks used in the Port Mansfield jetties were 5, 8, and 16 tons. Side slopes of the jetty mounds were approximately 1 vertical on 1 horizontal. A typical section of the jetty is shown in Fig. 2.

Subsequent to completion of the locally constructed improvements, extensive deterioration of both jetties occurred. In November, 1957 severe storms produced a period of very heavy seas and severe settlement of the jetties. Scouring currents washed away material at the inner ends of the jetties and resulted in shore line recession sufficient to flank both jetties. With the effectiveness of the jetties destroyed, extensive shoaling occurred in the channel between the jetties, which had been dredged to a depth of 16 ft. Field surveys were made during February, 1958 to determine the present condition of the improvements constructed by the local interests. No material changes were found in any of of the facilities except in the vicinity of the Gulf entrance, where the jetties and entrance channel were found to be badly deteriorated and not functioning so as to be useful to navigation.

No evidence was found of scattering or displacement of the tetrapods, but very extensive subsidence had occurred. Indications are that the subsidence of the jetties resulted from a combination of scour and compression of the base materials. Channels from 2 ft to 4 ft below the Gulf bottom had been scoured along the outer two-thirds of both sides of the north jetty and the north side of the south jetty. Available borings and probings that were made during the survey indicated that the base materials for the jetties consist of a sand layer, ranging from a few inches to several feet in thickness, overlying a soft plastic clay. Along the outer half of the north jetty, where subsidence had been most severe, no sand layer was found.

Subsidence affected the north jetty considerably more than the south jetty. Except for short reaches, the entire north jetty is now below the normal water surface with the settlement progressively greater toward the outer end, where the highest points on the tetrapods are from 3-1/2 ft to 7 ft below the water surface. From the midpoint of the north jetty, inshore for a distance of approximately 350 ft, a number of tetrapods remain above the water. In this reach a few inches of coarse sand were found overlying the soft clay. Considerably more sand was found along the base of the south jetty than along the north jetty. Although the entire south jetty has subsided, many of the tetrapods are still visible above the water over most of its length. Photographs of the visible portions of the jetties in February, 1958, seven months after completion, are shown in Fig. 3. Recent inspections indicate little recent subsidence and the jetties may have become stabilized, or they are subsiding at a very slow rate.

The high void ratio in the tetrapod jetties, which has been estimated at 50%, results in very low efficiency in intercepting and holding littoral drift materials and in confining tidal currents. When the jetties were outflanked by shore line erosion, their usefulness for any function, except as a breakwater, was destroyed, even though subsidence had not occurred. In the deteriorated condition, the jetties afford no protection to the dredged channel from a maintenance standpoint. This is evident from the rapid shoaling that completely obliterated the dredged channel seaward from about the original Gulf shore line. Moreover, without jetty protection, natural shore processes probably will tend to close this channel in the absence of major scour forces from storms or hurricanes.

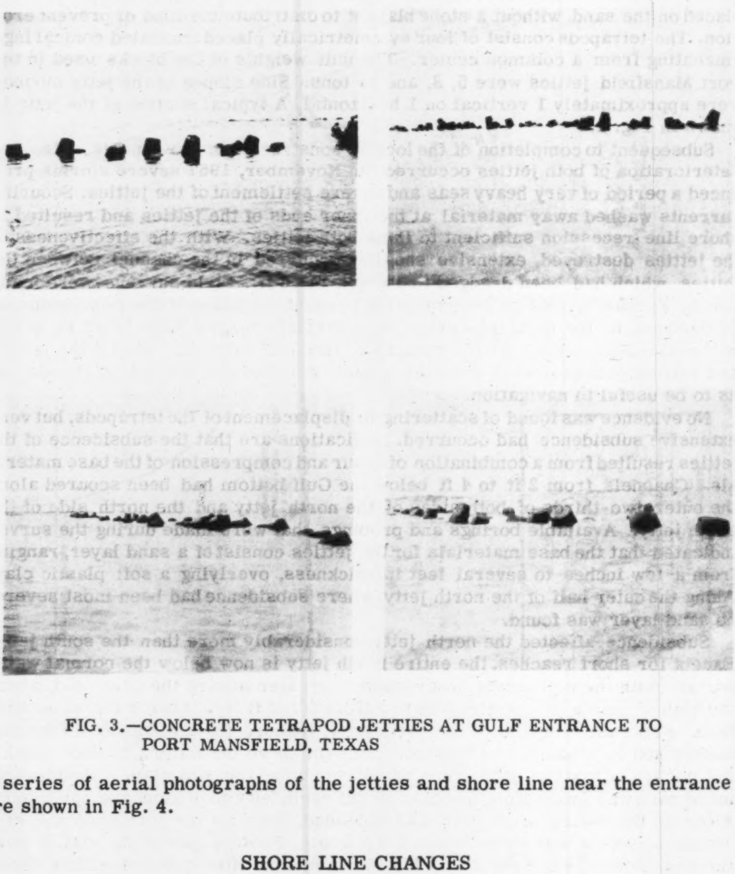

FIG. 3.—CONCRETE TETRAPOD JETTIES AT GULF ENTRANCE TO PORT MANSFIELD, TEXAS

A series of aerial photographs of the jetties and shore line near the entrance are shown in Fig. 4.

SHORE LINE CHANGES

The Gulf shore line north of the entrance and the channel banks for several thousand feet inland have undergone extensive erosion and accretion cycles since completion of the channel and jetties. The series of photographs in Fig. 4 and

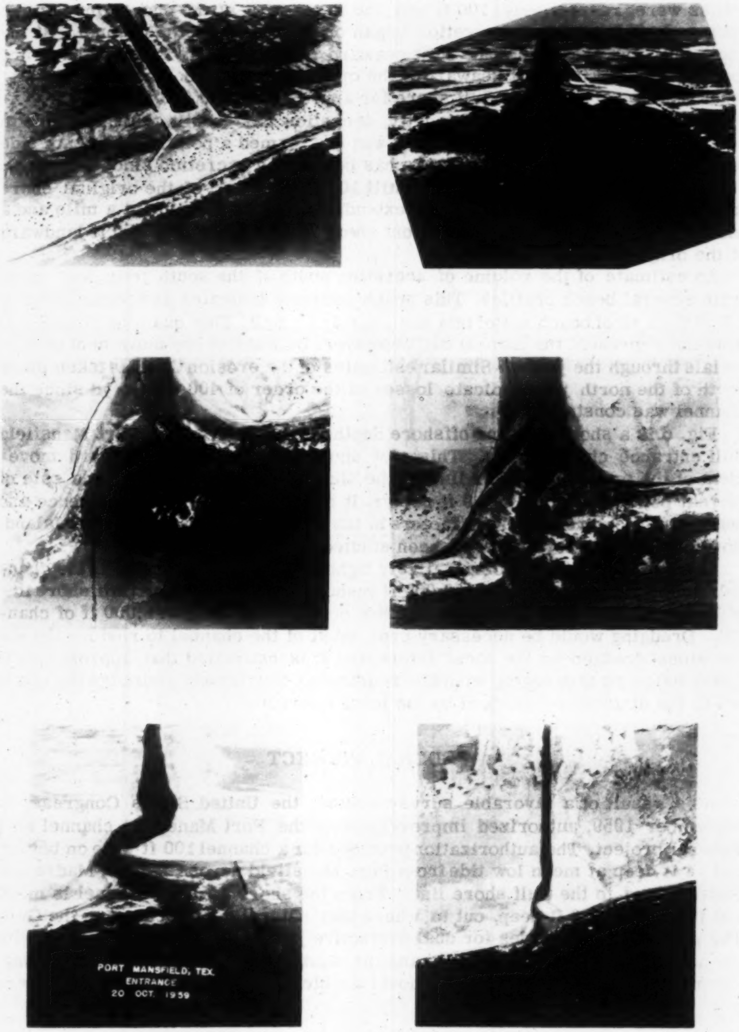
PORT MANSFIELD, TEX.
ENTRANCE
20 OCT. 1959

FIG. 4.—CONCRETE TETRAPOD JETTIES AND CHANNEL THROUGH PADRE ISLAND, PORT MANSFIELD, TEXAS

the comparative shore lines in Fig. 5 show these changes. Initially the width of the entrance widened from 250 feet to 1,800 feet and the shore ends of the jetties were widely flanked. In February, 1958 the shore ends of the north and south jetties were, respectively, 100 ft and 250 ft seaward of the shore line. In the spring of 1958, however, accretion began on both sides of the channel. At the south jetty the shore line has built seaward 700 ft from its farthest landward location and now is 250 ft seaward of the original shore line. The accretion extends southward along the Gulf shore for approximately 3,000 ft.

At the inner end of the north jetty, accretion has rebuilt the shore line to connect with the inner end of the jetty and has formed a point on the south side of the jetty. North of the jetty there has been some accretion since February, 1958 but the present shore line is still 100 ft landward of the original shore line. The erosion of the Gulf shore extends northward for almost a mile and a half and at its widest point the present shore line is more than 150 ft landward of the original shore line.

An estimate of the volume of accretion south of the south jetty was made from several beach profiles. This rough estimate indicates an accumulation of 250,000 cu yd of beach materials in a year and a half. This quantity of material does not represent the littoral drift however, because of the movement of materials through the jetties. Similar estimates of the erosion that has taken place north of the north jetty indicate losses of the order of 400,000 cu yd since the channel was constructed.

Fig. 6 is a shore line and offshore depth change study of the Port Mansfield Gulf entrance channel area. This plot shows the historical landward movement of the mean high water line on the Gulf side of Padre Island, the rate of retreat being approximately 9 ft per yr. It is possible that land subsidence and beach depletion are significant factors in the indicated losses from Padre Island. However, such factors have not been studied in detail.

Surveys indicate that comparatively light shoaling has occurred in the channels inland from a point about 2,000 ft inshore from the former Gulf shore line of Padre Island. Extensive shoaling has occurred in the last 2,000 ft of channel. Dredging would be necessary over most of the channel to restore the dimensions dredged by the local interests. It is estimated that approximately 1,000,000 cu yd of dredging would be required at this time to redredge the channel to the dimensions dredged by the local interests.

FEDERAL PROJECT

As a result of a favorable survey report, the United States Congress, in September 1959, authorized improvement of the Port Mansfield channel as a Federal project. The authorization provides for a channel 100 ft wide on bottom and 14 ft deep at mean low tide from Port Mansfield across Laguna Madre and Padre Island to the Gulf shore line. From the shore line the channel is made 250 ft wide and 16 ft deep, out to where that natural depth occurs in the Gulf. The project also provides for dual protective jetties at the Gulf entrance, with the north jetty to be 2,300 ft long and the south jetty to be about 2,270 ft long. Planning studies to determine the most feasible type of jetties to construct were

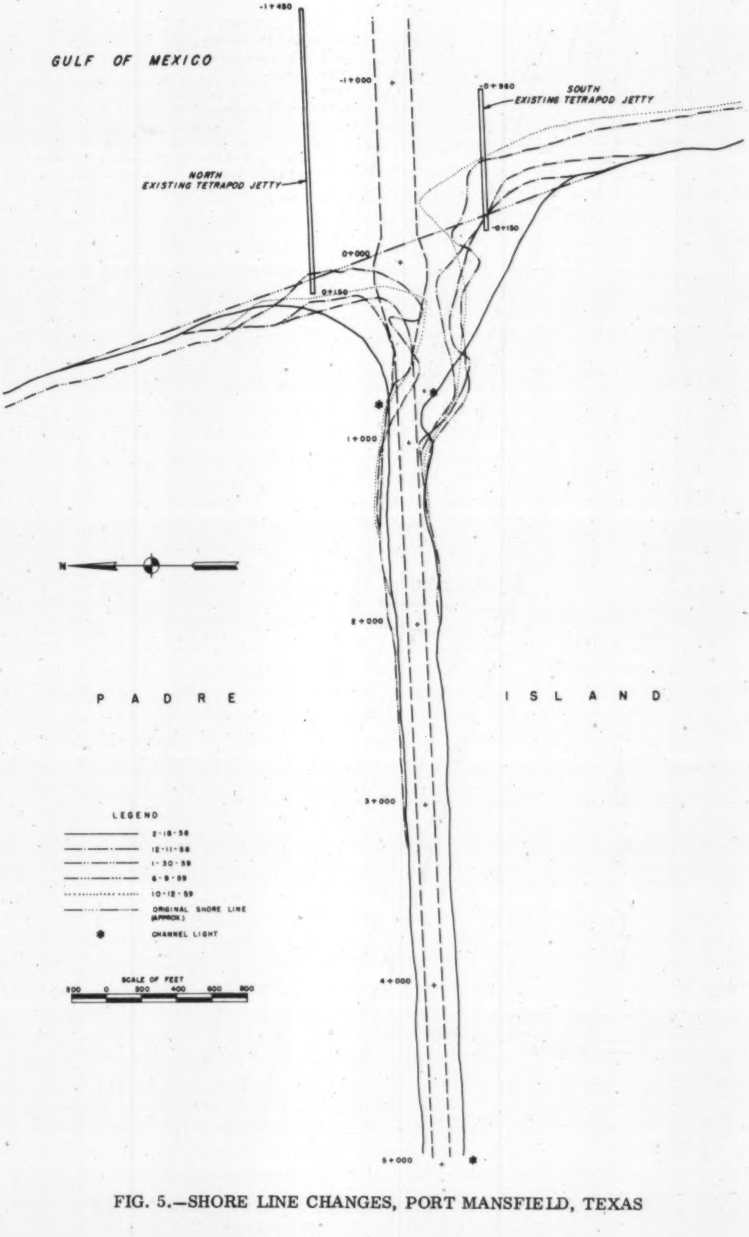
GULF OF MEXICO
-1+450
-1+000
-0+950
SOUTH
EXISTING TETRAPOD JETTY
NORTH
EXISTING TETRAPOD JETTY
-0+150
0+000
0+150
1+000
N
2+000
P A D R E
I S L A N D
3+000
LEGEND
2-18-58
12-11-58
1-30-59
6-9-59
10-12-59
ORIGINAL SHORE LINE (APPROX.)
CHANNEL LIGHT
SCALE OF FEET
200 0 200 400 600 800
4+000
5+000

FIG. 5.–SHORE LINE CHANGES, PORT MANSFIELD, TEXAS

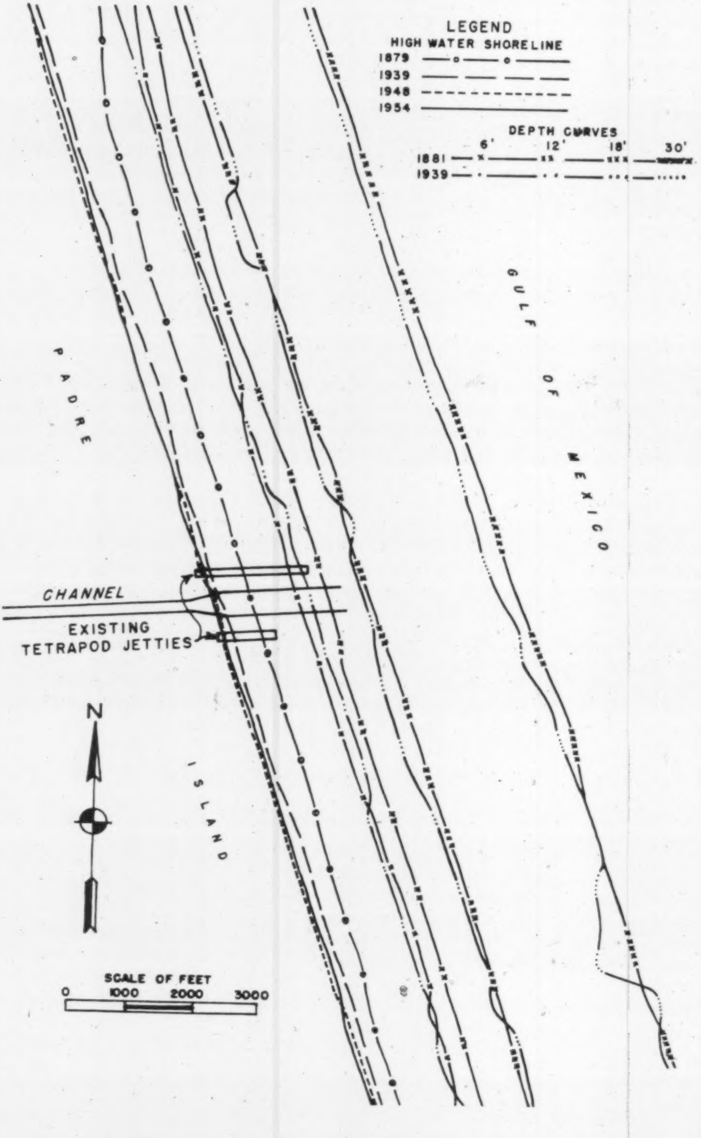
LEGEND
HIGH WATER SHORELINE
1879
1939
1948
1954
DEPTH CURVES
6'
12'
18'
30'
1881
1939
GULF OF MEXICO
PADRE ISLAND
CHANNEL
EXISTING TETRAPOD JETTIES
N
SCALE OF FEET
0
1000
2000
3000

FIG. 6.–SHORE LINE AND OFFSHORE DEPTH CHANGES, GULF ENTRANCE CHANNEL TO PORT MANSFIELD, TEXAS

completed on March 1, 1960. The data presented in the remainder of the paper are the results of the design studies.

JETTY LENGTH

Study of the most feasible types of jetties for the shallow-draft navigation entrance included consideration of the two principal functions to be performed by the jetties. These are (1) preventing the flow of littoral drift materials through or around the ends of the jetties into the navigation channel, and (2) acting as breakwaters to permit safe navigation of the entrance.

A relatively impervious and sand tight jetty is required to prevent the passage of littoral drift materials. Spacing of the jetties must be determined by consideration of the optimum distance for navigation and the location and lengths should provide desirable breakwater action and safe navigation through the entrance. The 1,000-ft spacing of the locally constructed jetties is considered to be satisfactory for these functions.

The length of the north jetty was determined primarily by the requirements considered necessary for breakwater action. The seaward end of the north jetty constructed by the local interests is approximately 1,300 ft from the original shore line, which was considered satisfactory from this standpoint. It was determined that the new north jetty for the 16-ft project entrance channel should have an overall length of 2,300 ft. Studies also indicated that for an 18-ft project channel, the north jetty should be extended to the 18-ft natural depth, which would require a seaward extension of 300 ft and an overall length of 2,600 ft. Similarly, for a 36-ft project channel, the north jetty should extend to the natural 30-ft depth in order to act effectively as a breakwater, and the overall length of the jetty for a 36-ft project should be 5,240 ft.

The length of the south jetty was determined by the anticipated accumulation of the littoral drift materials on the south side of the jetty and by the breakwater and navigation requirements. A study and analysis of the littoral drift along the Gulf shore of Padre Island was made by the Beach Erosion Board. This study indicated that there is considerable northward movement of beach materials along the shore that would accumulate against the south side of an impermeable jetty.

LITTORAL DRIFT

The study of beach characteristics at Port Mansfield and Brazos Santiago Pass was made to determine the effect of impermeable jetties at Port Mansfield entrance. The comparative mean high water line and offshore depth contours north of the north jetty at Brazos Santiago Pass indicate substantial loss of material from the beach and foreshore zones for a considerable distance to the north since initiation of the project (data covering the period 1881 to 1939 and 1948). This evidence strongly indicates predominant northerly littoral drift at a substantial rate. However, these data indicate only a minor amount of accretion to the south of the south jetty. It appears to be quite probable that a very large percentage of the littoral material movement to the north passes through the south jetty into the entrance channel. Maintenance dredging records of dredging for the Gulf bar and jetty channel indicate an average removal of 352,000 cu yd per year for the period from 1936 to 1957. Because the most likely major source of shoaling material appears to be littoral drift from the south, and as the erosion rate to the north supports this reasoning, net northerly rate of littoral drift at Brazos Santiago Pass was estimated to be 300,000 cu yd per year.

The general shore line at the Port Mansfield entrance channel has virtually the same orientation as at Brazos Santiago with comparable wave energy distribution. For design purposes the estimated 300,000 cu yd annual drift rate at Brazos Santiago Pass was considered to be applicable to the Port Mansfield site. Based on this estimate of littoral drift it was estimated that the accumulation would move the mean high water line seaward along the jetty over 10-yr periods as shown in Table 1 and in Fig. 7.

A study of the characteristics of the beach materials indicated that the outer end of the south jetty should be positioned to provide a 1-on-60 bottom slope from the mean high water line to the depth to be maintained in the jetty channel. With mean high water at elevation 2 ft above mean low tide and the recommended jetty channel depth of 16 ft below mean low tide, the outer end of the jetty should be 1,080 ft seaward of the mean high water line. This criterion would require a total jetty length of 2,270 ft to retain the littoral drift for a period of 10 yr. It is considered that for the 16-ft jetty channel, the south jetty should initially be built to contain a 10-yr accretion of littoral materials. This would afford a 10-yr period in which to study the actual rates of accretion of littoral materials and the effectiveness of the jetty in retaining the materials. Further, it is believed possible that development of the shore north of the north jetty over the next 10 yr may increase the land values so that bypassing the beach materials around the jetties may be warranted to prevent the erosion that would result from loss of littoral drift caused by the jetties. The south jetty would have an overall length of 2,270 ft. To provide satisfactory breakwater action for shallow-draft navigation, a jetty crown height of 5 ft above mean low tide is considered necessary.

TABLE 1.—LOCATION OF MEAN HIGH WATER LINE

Period after construction of the south jetty, in years	Length along south jetty, in feet
10	600
20	1,000
30	1,300
40	1,500
50	1,740

The length of the south jetty for deeper channels was also determined from consideration of the same factors. At an assumed angle of repose slope of 1-on-60 for the littoral materials, a depth equivalent to the entrance channel depth of 20 ft for an 18-ft project would be reached at a point 1,320 ft seaward from the high water line of the 10-yr sand accumulation south of the jetty. A jetty to that point would be 2,510 ft long. However, since the existing natural bottom-slope averages somewhat flatter than 1-on-60, a jetty length of about 2,930 ft would be required to reach the natural 18-ft depth contour. A length of 2,490 ft was considered to be the optimum length of the south jetty for a 20-ft entrance channel. This jetty would terminate in a natural depth of about 16-1/2 ft of water. Similarly, a 36-ft project with an entrance channel 38 ft deep would require a south jetty length of 3,590 ft to reach an equivalent depth of 38 ft on the assumed slope of 1-on-60. Because of the flatter natural bottom-slope, a total length of 5,130 ft would be required to reach the natural 30-ft depth contour. The latter length would be more desirable for furnishing breakwater protection for deep-draft

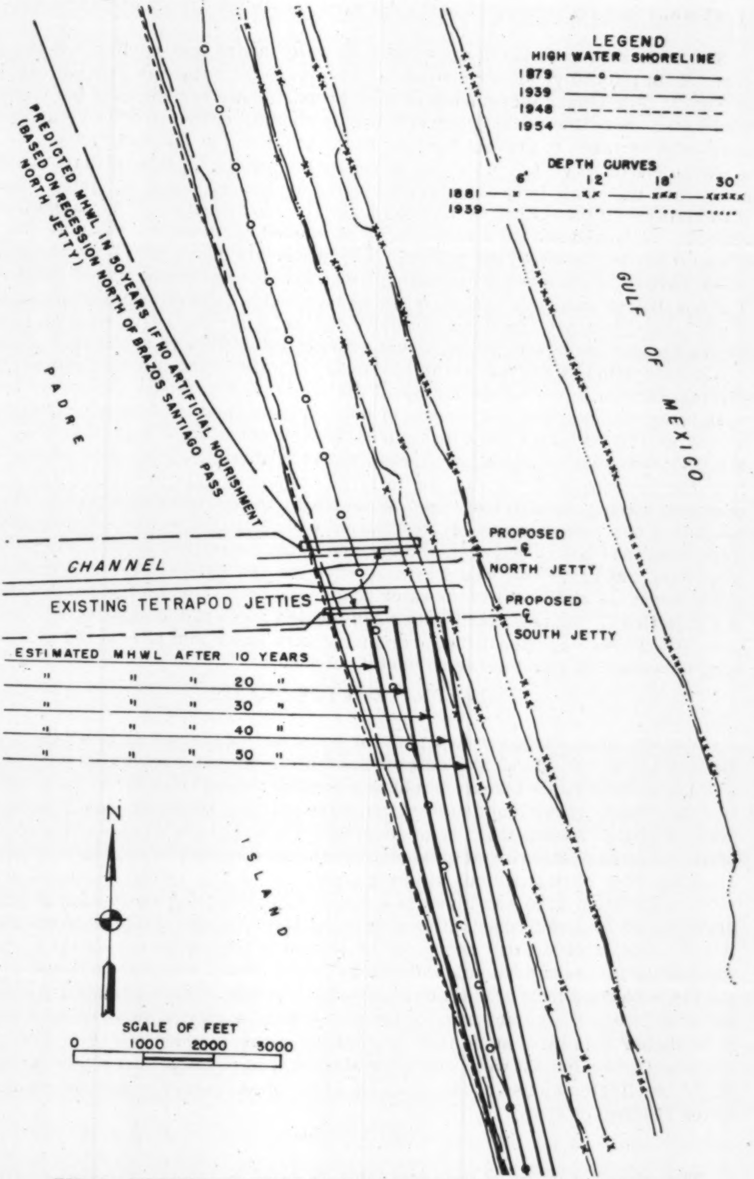
LEGEND
HIGH WATER SHORELINE
1879
1939
1948
1954
DEPTH CURVES
6'
12'
18'
30'
1881
1939
GULF OF MEXICO
PADRE
ISLAND
PREDICTED MHWL IN 50 YEARS IF NO ARTIFICIAL NOURISHMENT (BASED ON RECESSION NORTH OF BRAZOS SANTIAGO PASS NORTH JETTY)
CHANNEL
EXISTING TETRAPOD JETTIES
PROPOSED ℄ NORTH JETTY
PROPOSED ℄ SOUTH JETTY
ESTIMATED MHWL AFTER 10 YEARS
" " " 20 "
" " " 30 "
" " " 40 "
" " " 50 "
N
SCALE OF FEET
0
1000
2000
3000

FIG. 7.—PREDICTED SHORE LINE AND OFFSHORE DEPTH CHANGES

vessels and was considered to be the optimum length for a 38-ft entrance channel.

The estimated position of the mean high water shore line north of the proposed north jetty 50 yr after construction of entrance structure is also indicated in Fig. 7. For lesser increments of time proportionate recession of the mean high water shore line could be expected. It is considered that protective measures must be taken to prevent flanking of the inner end of the north jetty.

At the end of 10 yr, following completion of the jetties, the littoral drift would have filled the south jetty and started moving around the end of the jetty. The materials would then either be moved shoreward and deposited in the navigation channel, be moved seaward and lost, or be moved north across the entrance, where it would not affect the project. Any materials deposited in the channel would have to be removed by dredging to maintain the navigable project depths. The quantity of materials that might be so involved is extremely problematical. It was estimated that shoaling in the jetty channel, after the 10-yr interval following completion of the jetties, would average 100,000 cu yd annually.

Consideration was given to the possibility of raising the tetrapod jetties by placing additional materials on top of the concrete tetrapods. However, the possibility of continued settlement of the tetrapods and the lack of a suitable foundation for the jetties made an estimate of their condition at some future construction date highly speculative. Conservative estimating dictated the assumption that not enough of the existing tetrapods would remain to realize any appreciable saving through their use. Accordingly, the designs were made on the basis that new jetties would be constructed with their centerlines 50 ft southward from the existing jetty centerlines, and would not utilize the tetrapod jetty. Any of the tetrapods that would remain when the new jetties were constructed would serve as additional breakwater and scour protection on the north sides of the jetties. If the new south jetty were located north of the existing tetrapod jetty, it would be expected that any remaining tetrapods would be covered by sand accumulation and would serve no useful purpose.

ALTERNATE JETTY PLANS

Consideration was given to a plan for constructing the new jetties with prestressed concrete cylindrical cell piles, 54 in. in diameter with side thickness of 5 in. A stone filter blanket would be necessary along the jetty at the ground line to prevent scour from current and wave action. This plan was discarded because of anticipated difficulty in securing a sand tight structure and the probability of damage from impact and abrasion by movement of the protective stone along the base during periods of heavy seas.

Consideration also was given to a plan for constructing the jetties of solid prestressed concrete sheet piles with stone filter blanket erosion protection. It was found, however, that except in the shallow water reaches near the shore, the loading forces of accumulated sand and wave impact would require substantial stone backing to be placed along the wall to insure structural stability. Construction costs of a combination of the two types of construction were estimated to be higher than for a stone jetty without the concrete sheet piles. If suitable construction equipment were readily available, it is possible that slight savings might be effected by use of the concrete sheet piles onshore and in the shallow water reaches offshore.

JETTY DESIGN

The proposed design of the jetties differs from the design of existing jetties along the Texas coast in several aspects. One of these aspects is the positive features aimed at making the jetty impervious to the passage of littoral drift. The 3-on-1 side slopes are generally flatter than usual so that the size of the cover stone can be smaller. The cover stones are to be placed at random so that the face of the jetty will be rough and irregular. The design will permit the use of limestone as an alternate to granite for the cover stone. A cretaceous limestone of suitable quality and size can be obtained from quarries in the vicinity of Austin, Texas.

The structural design of the jetty is based on the results of analysis of the foundation soil, and the pressures on the jetty from design waves and tide heights.

Storm Tide and Waves.—The storm tide and wave heights used to design the jetty were determined from a consideration of the magnitude of hurricanes that can affect the lower Texas coast. The storm-tide height was based on the water level response at Port Mansfield that would result from a standard project hurricane crossing the coast on a path normal thereto and with its center south of Port Mansfield a distance equal to the radius of the region of maximum wind speed. The characteristics of the standard-project hurricane have been determined by the United States Weather Bureau, Dept. of Commerce (USWB) as follows:

CPI = Central pressure, 27.50 in.;

R = Radius to maximum winds, 13.5 nautical miles;

T = Forward speed of storm, 11 knots;

V_{cx} = Maximum cyclostrophic wind, 112 mi per hr;

V_{gx} = Maximum gradient wind (V_{cx} adjusted for coriolis affect), 110 mi per hr;

V_x = Maximum wind speed 30 ft above the water ($0.865\ V_{gx}$), 95.1 mi per hr;

W_m = V_x adjusted for direction and including $1/2\ T = 93.8$ mi per hr;

N_m = Wind set up of water at 6 fathom depth;

$V/\bar{c}$ = Shoreward velocity of storm divided by mean free wave speed;

F/L = Fetch divided by width of continental shelf;

S = Water level response factor, and

K = A constant based on known storm tide relationship, 1.62.

The formula, developed by the Beach Erosion Board in connection with hurricane studies, for determining the water level set up and using the above factors is:

$$N_m = K\ W_m^2\ S \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

The values used in this formula give storm tide level of 6 ft above mean low tide in the Gulf at Port Mansfield. The frequency of occurrence of the standard project storm is estimated at once in 100 yr.

A study of the possible wave heights that could be generated at the outer end of the jetties by the design storm indicates that the depth of water would control the wave height, because the maximum wave that would break in the available water depths could be generated by the design storm. The maximum wave, that is the average of the highest 10% of the waves in the wave spectrum, is estimated at 12 ft for the standard project storm at the outer end of the jetty. The wave would be reduced to 4 ft at the shore line because of the shallow water.

Foundation Investigations.—In December, 1959 and January, 1960 fourteen undisturbed borings were made along the proposed new jetty alignments. Seven borings were drilled on land and seven in the water. The logs of the borings are shown in Fig. 8.

The shore end of the jetties will rest on approximately 5 ft to 7 ft of fine sand. The sand strata decreases in thickness seaward and is absent from the foundation areas of the outer ends of the jetties. Occasionally sand is deposited by the shore currents over the soft clay beneath the outer ends of the jetties. Below the sand strata on the shore end of the jetties is a medium stiff clay lense 6 ft to 8 ft thick with a soft clay strata 10 ft to 12 ft thick below it that extends the entire length of the jetties. Below the soft clay strata a 3 ft to 4 ft thick soft clayey to silty sand strata is encountered. Below the soft clayey or silty sand strata, hard clays and dense sands are encountered to a depth of at least 50 ft below sea level.

Moisture content, liquid limit, and grain size distribution tests were made on all samples. Density tests were also made on all undisturbed samples. Consolidated-drained and unconsolidated-undrained shear tests were made on typical samples from the various foundation strata. Unconsolidated-undrained triaxial shear tests were performed on typical foundation materials to check the test results obtained from the direct shear tests. Consolidation tests were also performed on the typical foundation materials. Composite consolidation curves for the typical foundation materials were drawn using the average of several curves made on each material.

The soft clay strata is about 12 ft thick beneath the outer ends of the jetties. The average moisture content is about 50% and the unit dry weight is about 72 pct. Both consolidated-drained, and unconsolidated-undrained direct shear tests were made on the soft clay strata. Triaxial unconsolidated-undrained shear test results were used to check the direct shear test results for final design. Values used for design were selected from these results.

Jetty Section.—Studies based on the data obtained from subsurface exploration and testing indicate that a rubble mound jetty can be constructed at the Gulf entrance to the Channel to Port Mansfield without excessive settlement. The section has been designed to meet the following major requirements: (a) Prevent flow of littoral drift material through the jetty; (b) Be stable against a design wave height of 12 ft; (c) Be safe from shear failure in the soft foundation material; and (d) Reduce wave action between the jetties to a minimum.

The jetty section that most adequately meets the above requirements would have a relatively impervious core with successive layers of larger stones to prevent piping. The side slopes must be flat enough to prevent foundation failure. A jetty section consisting of the featuring elevation of the top of jetty of $+$8.0 ft MLT, top width of 16 ft, and side slopes of 1-vertical-to-3-horizontal is proposed. A stone blanket 3 ft thick will be provided under the entire jetty and extending from 2 ft to 15 ft beyond the toe. This blanket will be composed of crushed stone 1/2 in. to 200 lb in size. Typical sections of the jetty are shown in Fig. 9.

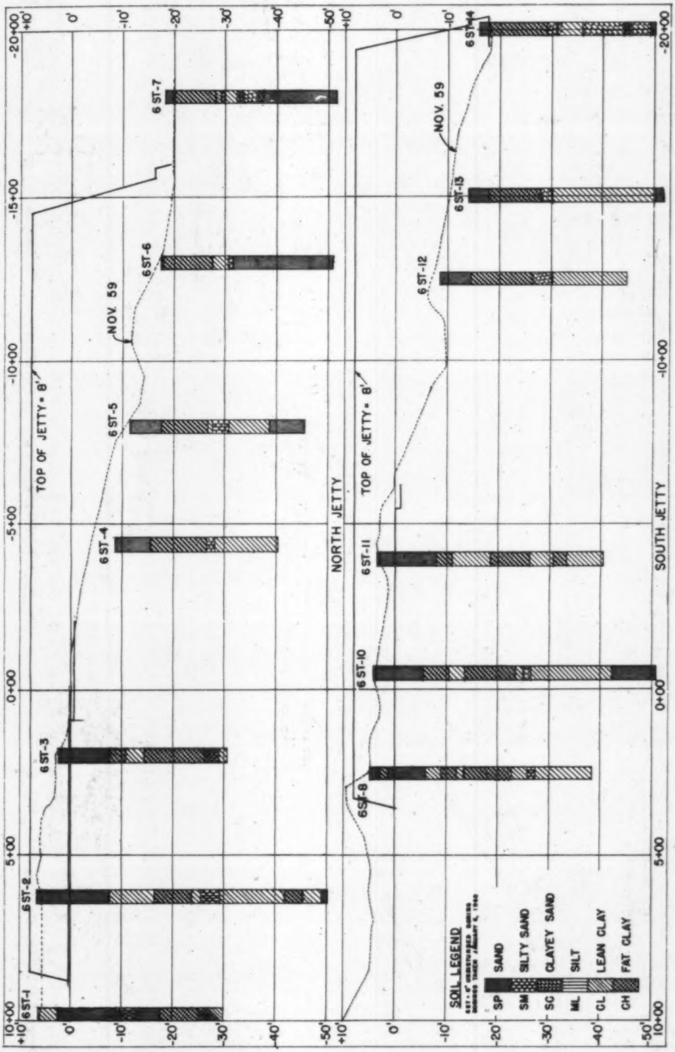
NORTH JETTY
SOUTH JETTY
TOP OF JETTY = 8'
TOP OF JETTY = 8'
NOV. 59
NOV. 59
6 ST-1
6 ST-2
6 ST-3
6 ST-4
6 ST-5
6 ST-6
6 ST-7
6 ST-8
6 ST-10
6 ST-11
6 ST-12
6 ST-13
6 ST-14
10+00
5+00
0+00
-5+00
-10+00
-15+00
-20+00
+10'
0'
-10'
-20'
-30'
-40'
-50'
SOIL LEGEND
SP SAND
SM SILTY SAND
SC CLAYEY SAND
ML SILT
CL LEAN CLAY
CH FAT CLAY

FIG. 8.—BORINGS AND JETTY PROFILES

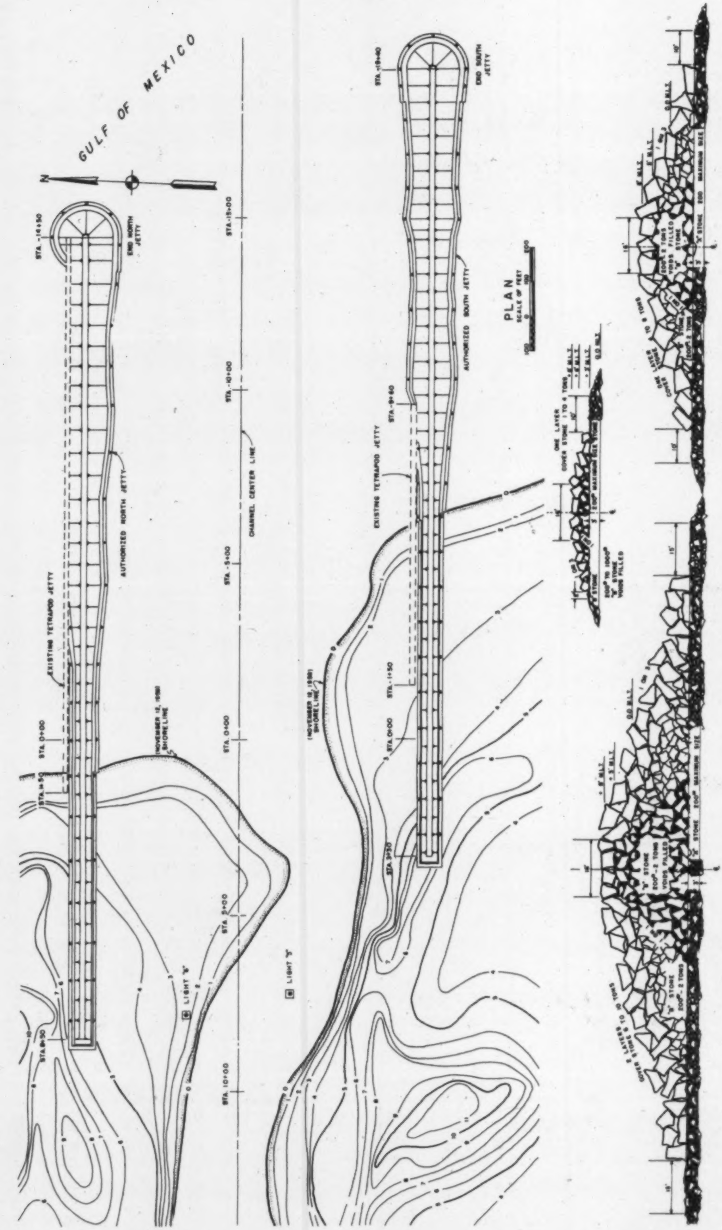
GULF OF MEXICO
N
STA.-14+50
END NORTH JETTY
STA.-15+00
STA.-10+00
CHANNEL CENTER LINE
AUTHORIZED NORTH JETTY
STA.-5+00
EXISTING TETRAPOD JETTY
STA. 0+00
SHORE LINE
STA. 5+00
LIGHT "E"
LIGHT "F"
STA. 10+00
STA.-19+40
END SOUTH JETTY
AUTHORIZED SOUTH JETTY
STA.-9+60
EXISTING TETRAPOD JETTY
STA.-1+50
STA. 0+00
PLAN
SCALE OF FEET
ONE LAYER COVER STONE 1 TO 4 TONS
200# TO 1000# "B" STONE VOIDS FILLED
"A" STONE
200# MAXIMUM SIZE
0.0 MLT.
COVER STONE 5 TO 10 TONS
"B" STONE 200#–2 TONS VOIDS FILLED
"A" STONE 200# MAXIMUM SIZE
ONE LAYER COVER STONE 1 TO 8 TONS

FIG. 9.—TYPICAL JETTY CROSS SECTION AND PLAN

The core will consist of reasonably graded stones from 200 lb to 4,000 lb in weight. In order to provide a relatively impervious core that will prevent flow of sand through the jetty, voids in the center section will be filled with 1/2 in. to 4 in. material. The top of the core will be constructed to + 5.0 ft MLT to provide an elevation that will be above the area subject to littoral drift after expected settlement of the foundation and in the jetty section due to rearrangement of stones. Smaller core stones ranging from 200 lb to 1,000 lb will be used on the minimum size section constructed on land.

Cover stones for the section of jetty extending from the shore line at elevation 0.0 MLT to the 10-ft depth will be one layer of 2-ton to 6-ton stones and, from the 10-ft depth to within 100 ft of the outer end there will be one layer of 4-ton to 8-ton stones. On the outer 100 ft of jetty and the end section 2 layers of 6-ton to 10-ton stones will be used. Cover stones for the section of jetty extending from the shore line at elevation 0.0 MLT to the inner end will be 1-ton to 4-ton stones.

The centerline of each jetty will be located 50 ft south of the centerline of the existing tetrapods.

The size of cover stones has been determined using Irribarrens formula as modified by Hudson. The average specific gravity of limestone investigated is 2.53 whereas that of the granite is 2.63. A design wave height of 12 ft at the outer end of the jetties has been used. As the wave height will decrease as the shore is approached, smaller stone are used for the land sections.

An analysis of the jetty and foundation stability was made by the modified Swedish Slide method and by Jurgenson's method. The modified Swedish Slide method gave the least safety factor. The section of the jetty deemed to be most nearly critical was at the outer jetty ends where the height was at a maximum and the base of the jetties rests on the soft clay strata. The soft clay strata is approximately 12 ft thick with strong sand and clays below. Unconsolidated-undrained direct shear tests give average values for the soft clay strata of 0.25 tons per sq ft for the cohesion and 5.2° for the angle of internal friction. Consolidated-drained direct shear tests give average values for the soft clay strata of 0.17 tons per sq ft for the cohesion and 23.3° for the angle of internal friction. The unconsolidated-undrained triaxial shear tests on this material give average values of 0.275 tons per sq ft for the cohesion and 0.0° for the angle of internal friction. The safety factors using test results from the three different types of shear tests are all adequate. Analysis using results from the unconsolidated-undrained triaxial shear tests gives the lowest safety factor.

The majority of the expected foundation settlement will be due to consolidation of the soft clay strata. The settlement has been computed using the consolidation curves for this material. The computed foundation settlement due to consolidation of this soft clay strata ranges from a negligible amount on the shore ends to a maximum of 0.75 ft at the outer ends of the jetties. The total settlement of the jetty due to the consolidation of the foundation and jetty materials is expected to be of the magnitude of 3 ft.

CHANNEL SHOALING

Surveys of the channel in the 2 1/2 yr since its completion have been made on eight occasions. These surveys varied from centerline profiles by fathometer to cross sections with sounding lead. A study of the results indicates that following the initial erosion at the Gulf entrance when there apparently was erosion in a large part of the channel, shoaling has occurred throughout the channel. The average shoaling increased from about 4 cu yd per ft in the 14,000-ft reach east of the Intracoastal Waterway, to 8 cu yd per ft across the shallow reaches of Laguna Madre, and about 16 cu yd per ft in the 6,000-ft reach west of the Gulf shore line. The estimated shoaling in the channel between the Intracoastal Waterway and the Gulf shore amounted to about 300,000 cu yd annually.

TIDE OBSERVATIONS

Recording tide gages have been operated at four locations on the channel since February 1958. The gages were located at the Gulf entrance, the Gulf Intracoastal Waterway and two intervening locations on Padre Island. Difficulties with the gage mechanism prevented continuous recordings and, because the rapid shoaling on the bar affected the tidal flow into the channel, firm conclusions on tidal flows in the channel cannot yet (1960) be made.

It has been established, however, that the water level in the bay is generally 1 ft to 2 ft higher than in the Gulf, except during short periods of the normal high tide cycles in the Gulf and during abnormal high tides induced by storm periods in the Gulf. The normal range of tide cycles in the bay is insignificant, being generally on the order of 0.1 ft to 0.3 ft. Sustained periods of south winds cause a slow, prolonged rise in the water level of the bay, which is equally slow to recede after the winds have subsided. During these periods the water level may be raised from 1 ft to 2 ft above normal during a period of two weeks or longer.

The observed currents in the small, meandering channel, which is being maintained at the Gulf entrance by natural forces, have been outbound practically the entire time. Velocities through the constructed reach of the channel range from slight to moderate, depending mostly upon the water level in the Gulf. On rare occasions, high tides in the Gulf will result in a reversal of the current for short periods of time. Farther inland, where the channel section has not been reduced appreciably by shoaling, current velocities are barely noticeable. Based upon the incomplete data, it appears that when the jetties at the entrance are reconstructed and the shoaled area has been removed, outbound currents of slight to moderate velocity should prevail in the channel most of the time. No conclusion can be formed at this time (1960) as to whether or not the entrance will be virtually self-maintained.

ACKNOWLEDGMENTS

The conclusions in this paper are based on studies made by personnel in the Engineering Division of the Galveston District and the staff of the Beach Erosion Board of the United States Army Corps of Engineers, in the design of the jetties and channel at Port Mansfield, Texas, under the civil works construction program of the Corps of Engineers. Data on the design hurricane are from publications of the USWB. The permission granted by the Chief of Engineers to publish this information is appreciated.

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TRANSACTIONS

Paper No. 3174

FREEWAY SPACING IN AN URBAN FREEWAY SYSTEM

By James M. Peterson,[1] M. ASCE

SYNOPSIS

The various controls that influence freeway spacing in an urban freeway system are described and categorized. A simplified study procedure is outlined that illustrates the interrelationships of certain variables affecting freeway spacing in a grid system.

It is estimated that by 1975, two-thirds of our rapidly mushrooming population will be concentrated in less than twenty super-urban regions.[2] Attending this growth and concentration of population will be great changes in governmental administration, social and economic activities, land use planning and communication. Of all the forces shaping these changes, the most important are those based on transportation needs. To the planning engineer has fallen the task of providing a circulation system that will adequately handle the travel linkages between the various land uses for today's needs and those of the foreseeable future.

To adequately provide for the mounting volume of vehicular traffic, any large urban area requires some form of a freeway system, be it rectangular, radiating, circumferential, or a combination of these. It is axiomatic to say that this system should be well integrated with the local street system in order to provide optimum traffic service. Regardless of the configuration of the system or the spacing of the freeways within it, these freeways will fill as the years pass. However, as the freeway system develops, the planning engineer

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[2] U. S. News and World Report, November, 28, 1958, Interview with Philip M. Hauser.

is constantly concerned with whether or not the individual freeways are properly spaced to provide optimum traffic service and also permit orderly community development. The amount of traffic that will be diverted to a freeway can be predicted by expressing percent users as a function of time and distance savings or freeway-city street time ratios. This is just one of the factors required to solve the complex problem of freeway spacing in an urban freeway system.

In recent years, there has been an increasing number of cities that have based their freeway-system needs on area wide transportation studies. These studies include the assignment to a system of future traffic based on population distribution, land use pattern, trip generation, and interzonal traffic volumes. In the Chicago Study and elsewhere a great deal of work is being done to develop computer programs that take into account all the variables affecting travel in an urban area, including such factors as motor vehicle ownership, trip length distribution, employment, population distribution, land use, trip purpose, and travel times. Analyses of this type permit systematic adjustment and retesting until the freeway system exhibits maximum traffic-service characteristics. It seems appropriate, before embarking on such an extensive program, to develop an appreciation of the various basic controls involved in freeway spacing in an urban freeway system.

Physical controls on freeway spacing are those imposed by the distance requirements for geometrics and signing. These are the distances necessary to develop reasonable grades, permit safe merging and weaving, and allow the motorist sufficient recognition and decision time at interchanges and ramps.

The distances shown in Fig. 1 are desirable minimums and result in a spacing between freeways of about 2 miles. Note that this spacing allows only one full set of ramps from the city streets connecting to the freeway between freeway interchanges. As a rule it will be necessary to increase this spacing 1 mile for each additional full set of ramps. Thus a freeway system based on 4-mile spacing could provide three complete on and off movements from the freeway to the city streets. Fig. 1 is also based on the desirable design practice of having no ramps to the city streets within the area of an interchange between two freeways.

The three cases shown in Fig. 1 reflect typical ramp patterns for handling design peak hour traffic desires. Case 1 is a four quadrant cloverleaf with 200-ft-radius loops and a system of collector-distributor roads. It is assumed that the collector-distributor road enters the freeway as a single lane on-ramp. This limits the design peak-hour volume entering the freeway from one direction to about 1,500 vehicles (the capacity of one freeway lane). This is approximately the number of vehicles that could be expected to come from a city street that carries, on the average, about 500 vehicles per hour per lane.

Case 2 illustrates a typical full diamond that would be used in locations where the design peak-hour-volume entering the freeway in one direction does not exceed 1,000 vehicles per hr. Case 3 shows a method for doubling the number of connections to the city streets without increasing the number of connections to the freeway over four. A design such as this would be required in an area where the individual street capacities are not high enough to permit picking up the traffic at one point such as in Cases 1 and 2. The design peak hour volume entering the freeway in one direction for Case 3 would be 1,500 vehicles. Again, this is the practical capacity of a single freeway lane and represents about all the vehicles that could be drawn from two average city streets.

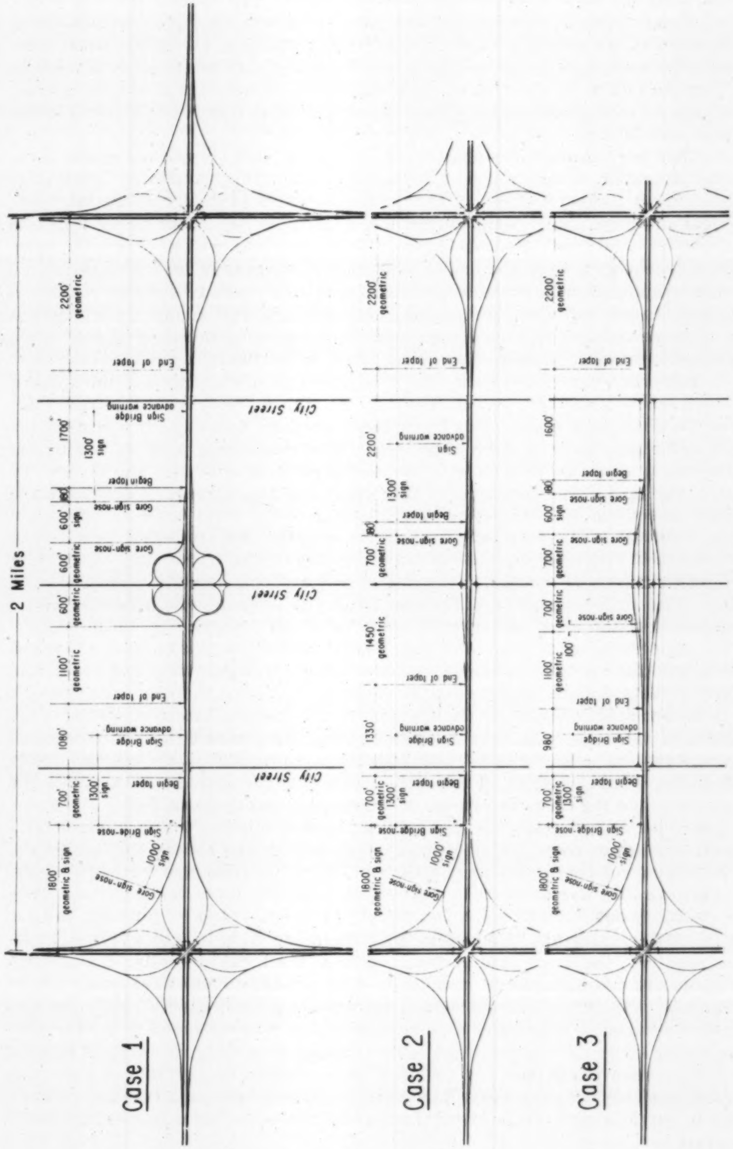
2 Miles
Case 1
Case 2
Case 3
City Street
City Street
City Street
2200' geometric
End of taper
1700'
Sign Bridge advance warning
1300' sign
Begin taper
180'
Gore sign-nose
600' sign
600' geometric
600' geometric
1100' geometric
1080'
700' geometric
Sign Bridge-nose
1800' geometric & sign
1000' sign
Gore sign-nose
2200'
Sign advance warning
700' geometric
1450' geometric
1330'
1600'
700' geometric
100'
980'

FIG. 1.—FREEWAY SPACING PHYSICAL CONTROLS

The existing master-planned city street pattern will control freeway spacing to some degree. The city street pattern affects the type and number of ramp connections which, in turn, affect freeway spacing. Of the practical controls on freeway spacing most important is probably the availability of funds in the planned future for the freeway system. There is no point in beginning construction, or even planning for a freeway system that cannot be financed in the foreseeable future.

Another very practical consideration affecting freeway spacing is the variability of traffic desire found in any urban area. This variability is directly related to the intensity of use of the main categories of land, residential, commerical and industrial. Trip generation density is dependent on the type and intensity of land use. Hence, highly developed areas will generate the most trips and require closer spacing of freeways than areas of less growth.

Topography often imposes definite controls on a freeway system. Location problems caused by bays, peninsulas, and hills are well known. Circumvention of these natural barriers can usually be justified because of excessive construction costs and may often be justified on the basis of traffic service.

An important practical control from an operational standpoint is the balance between freeway and city street service. All trips result from the travel desires between land uses. The basic function of the freeway is to provide for most of the movement portion of the trip. The city street permits fulfillment of the trip purpose by providing terminal distribution of traffic and direct access to the land. The city streets also serve the short trips. If freeways and ramp connections are too close together, many of the short trips are attracted to the freeway. The resultant congestion impairs the freeway's function of movement. If freeways and connections are too far apart, too many intermediate length trips will remain on the city street system, causing congestion and delay. The proper balance of freeways and city streets permits each system to supplement the other and operate at optimum efficiency. The optimum condition represents a delicate and often not too clearly defined balance between trip length distribution, trip generation density, trip purpose, and capacities of city streets, ramps, and freeways.

In recent studies made in California for the establishment of a state-wide system of freeways and expressways, it was determined that the completed freeway system in the greater Los Angeles area would carry approximately 52% of the vehicle miles. This is in contrast to the more rural areas of the state, in which the freeway system is expected to carry about 70%.

There are many intangible controls on freeway spacing which assume real significance when developing a freeway system that will receive the backing of both the general public and local officials. These controls are termed intangible because they are difficult to express as concrete facts and figures. They are probably best described as community benefits. Loss of tax base falls in the preceding category. Unquestionably the more right of way a freeway pattern requires, the more land is removed from the local tax rolls. Local communities are understandably concerned with possible loss of tax base. However, it is considered that this loss of tax base is principally an imaginary one. Surveys conducted in California do not indicate a reverse trend in over-all tax base even in small communities where millions of dollars worth of right of way have been purchased for freeway use. Furthermore, it has generally proved true that over the years land values appreciate near a freeway system. This is particularly true in areas that can be developed for commercial or industrial use.

Removal of productive capacity caused by right of way acquisition is another undesirable immediate affect of the freeway. However, obviating the need for a freeway by excessive removal of trip producing land is quite unlikely. While the removal of commercial, industrial, or residential land may be undesirable in the short run, it is usually compensated for by more intensive use of the land adjacent to the freeway.

The economy of a freeway creates an intangible control on freeway spacing. It is a generally accepted tenet that the money invested in construction of a freeway must develop over a period of years a return to the highway user who pays for it. This return is usually termed user savings.

The computation of highway user savings is based on the value of savings that will accrue to the motorist (usually over an amortization period of 20 yr) in reduced travel time, and reduced operating costs by utilizing the proposed freeway facility in lieu of existing surface streets. Monetary values have been developed for computing user savings. For automobiles freeway operation is often rated at 4.5¢ per mile as compared to 4.75¢ per mile on conventional highways and city streets. The figure 2.6¢ is often assumed to be the value of a passenger-vehicle minute. The difficulty in properly defining the worth of short increments of time saved over a 20 yr period leads us to place user savings as a whole in the intangible category. User savings are computed for various freeway location lines, and the best line, other factors being equal, is usually considered to be the one with the largest user savings. Thus, in a sense, user savings may have considerable influence on the location of a freeway.

Comfort and convenience are intangibles that influence a freeway system. The extent to which the public will support a freeway program is a measure of how much the motorist is willing to pay for comfort and convenience of travel. There is little doubt that the motorist is willing to pay, but how much is difficult to evaluate.

From a traffic service standpoint the ideal approach to the freeway spacing problem would be to test and retest various configurations of freeways for an entire urban area using the latest parameters of travel and the most recent computer programs. Mention of such an undertaking is beyond the scope of this paper. However, on a theoretical basis, it is possible to show the relationships between freeway spacing, freeway access spacing, trip length distribution, and vehicle speeds. It is also possible to derive the relative traffic service provided by the freeway system and the city street system. An analysis of this type using various freeway and freeway access spacings has been partially completed (as of 1960) by the California Division of Highways. A general description of the study procedures and the results based on 4 mile spacing of freeways will serve to illustrate the relationships previously mentioned.

The problem is to compute the number of automobile trips generated in a selected area and determine the number that would use a grid-type freeway system with 4 miles between freeways.

Solution and Assumptions.—

Step 1.—Select a 16 sq mile area and assume that the freeways running N-S and E-W cross at the center of the area, dividing it into NE, SE, SW, and NW quadrants.

Step 2.—Assume a spacing for access points from the city streets to the freeways with no access points at the center of the area where the two freeways interchange.

Step 3.—Assume or develop constants for (a) freeway and city street speeds, (b) population of the area, (c) land use of the area, (d) number of dwelling units, (e) average daily residential trip origins per dwelling unit, (f) average daily vehicle trip origins per net acre of commercial land, (g) same as (f) for industrial land, (h) overall trip length distribution (airline), and (i) time-distance assignment curves to determine percent of freeway users for each trip length category.

Step 4.—From these constants, compute the number of vehicle trip origins in the 16 sq miles and assume they are evenly distributed over the area and that desire for travel is equal in each octant of a circle.

Step 5.—Assume that the trip origins are concentrated in a number of zone centroids.

Step 6.—Analyze the zones in one quadrant and apply the results to the other three quadrants by identity.

Step 7.—Develop percentage freeway assignment curves for parallel and diagonal users. (Parallel includes all directions from 22-1/2° on one side to 22-1/2° on the other side of N,S,E, or W. Diagonal includes all directions from 22-1/2° on one side to 22-1/2° on the other side of NE,SE,SW, or NW).

Step 8.—With the percentage freeway assignment curves assign the freeway users (Parallel and diagonal) from each quadrant to the proper freeway access points.

Step 9.—Compute the one-way annual daily traffic (ADT) on one freeway by totaling the assigned trips at successive access points. Four times this figure is the total one-way ADT leaving a 16 sq mile area.

The example to follow demonstrates the procedure outlined. All of the computations for this example were performed manually, but it should be noted that the entire process could be programmed for high speed computers utilizing, in part, the "shortest path through a maze," technique.[3] Storage of assignment curves and actual assignment to a network has also been programmed for computers.

EXAMPLE

Step 1.—The general area selected is an incorporated suburb in the greater urban area, chosen for its general homogeneity and location on comparatively level terrain. The area is, for the most part, composed of middle class residential neighborhoods with typical commercial development. There is no industrial land use. The N-S, E-W freeways were assumed to cross in the center of the 16 sq mile study area.

Step 2.—Access points were located at mile intervals (Fig. 2). The street pattern was assumed to be a grid, parallel to the freeway grid, with major streets at mile intervals and secondary streets on the 1/4 mile and 1/2 mile points. It was assumed that this system of streets and freeways extended, without limit, in all directions.

Step 3.—(a) Assumed speeds were 52 mph on the freeways and 25 mph on the city streets. (b) The population data were obtained from the local county Regional Planning Commission and averaged 8,859 persons per sq mile. (c) The commercial land use was determined on the basis of 2.6 net acres of commercial land need for each 1,000 persons. There was no industrial land. (d) The number of dwelling units was obtained by assuming three persons per dwelling unit and dividing this into the 8,859 persons per sq mile giving 2,953 dwelling units. (e) Daily residential vehicle trip origins per dwelling unit were taken as four, based on generation studies by the local county road department. Residential trip origins per square mile then equals (2,953) (4) = 11,812. (f) The average daily vehicle trip origins per net acre of commercial land were taken as 250. This is comparable to figures developed in San Diego, California and Detroit, Michigan. The total commercial origins per square mile is $\left(\frac{8,859}{1,000}\right)$ (2.6) (250) = 5,758. (g) No industrial origins were used in this residential area. (h) Percentage trip length distribution (airline) was taken from a motor vehicle use study conducted by the California Division

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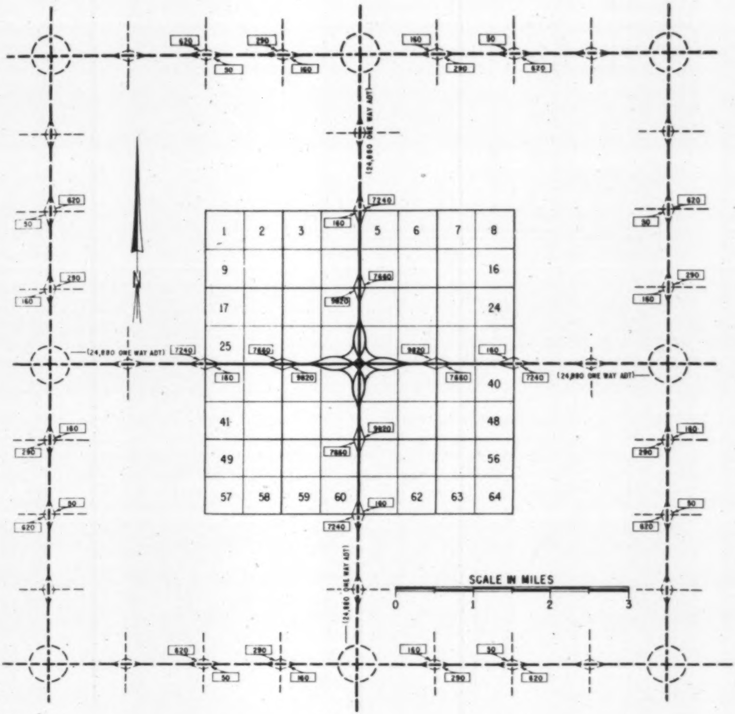
[3] "The Shortest Path Through a Maze," by Edward F. Moore. Presented at the 1957 Internatl. Symposium on the Theory of Switching at Harvard Univ., at Cambridge, Mass.


620
90
290
160
160
290
90
620
(24,880 ONE WAY ADT)
7240
160
7660
9820
N
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(24,880 ONE WAY ADT)
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(24,880 ONE WAY ADT)
SCALE IN MILES
0
1
2
3
620
50
290
160
160
290
50
620

FIG. 2.—THEORETICAL TRAFFIC GENERATION FROM A 16 SQ MILE AREA

of Highways for the local urban area. (i) the "Percent of Traffic Diversion to Freeway" curves (Fig. 3) used by the California Division of Highways were used in comparing the city street trips with freeway trips.

TABLE 1.—PERCENTAGE DISTRIBUTION OF TRIPS (AIRLINE)

Mile	Percent	Mile	Percent	Mile	Percent
.5	10.29	11	1.93	22	0.35
1	20.55	12	1.63	23	0.31
2	14.00	13	1.39	24	0.27
3	9.98	14	1.17	25	0.23
4	7.54	15	1.00	26	0.22
5	5.97	16	0.86	27	0.19
6	4.80	17	0.72	28	0.17
7	3.92	18	0.64	29	0.15
8	3.28	19	0.54	30	0.13
9	2.74	20	0.47	> 30	1.87
10	2.28	21	0.41		

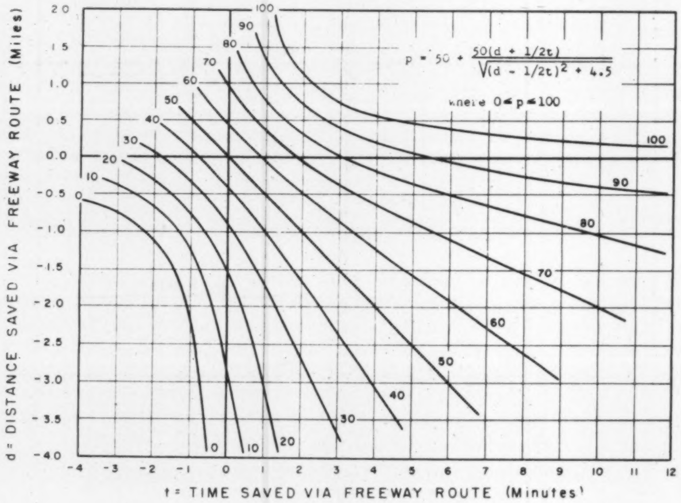

FIG. 3.—PERCENT OF TRAFFIC DIVERSION TO FREEWAY

Step 4.—The total daily vehicle trip origins per square mile was 11,812 + 5,758 = 17,570. This figure was rounded to 18,000 for the actual assignment computations. The 16 sq. mile area generated a total of 288,000 trip origins. The assumption of even distribution and equal travel desire was made.

Step 5.—The 16 square miles were divided into 64 quarter square mile zones (Fig. 2). This zone size was chosen to minimize centroid bias and still keep the number of calculations within the limit of manual operation.

Step 6.—Since it was assumed that the origins were evenly distributed and there was equal desire for travel in all directions, only the zones in one quadrant were analyzed. These zones, shown in Fig. 4, were divided into three classifications according to their distances from the freeway. In other words, 43.75% of the zones were in the tier of zones immediately adjacent to the freeway, 31.25% were in the next tier away, and 25% were two tiers away. In this case, the NE quadrant was analyzed and the results applied to the other three quadrants by identity.

Step 7.—The percentage freeway assignment (Fig. 4) curves for parallel and diagonal users for the three classifications of zones were developed for the NE quadrant as follows: (a) Radial lines were drawn from each centroid to include 22-1/2° on either side of E and W for parallel users and 22-1/2° on either side of NE and SW for diagonal users. (b) With each centroid as a center, arcs were swung intersecting the radial lines at the half-mile points for a distance of 15 miles. It was found that trips over 15 miles had a 100% assignment to the freeways. (c) The time and actual distance were computed for the city street route and the freeway route for each zone included in a 45° sector of each mile ring. This was done for distances up to 15 miles for E and W directions (parallel) and NE and SW directions (diagonal). The other directions were assumed to be the mirror image of the first four. By taking the difference

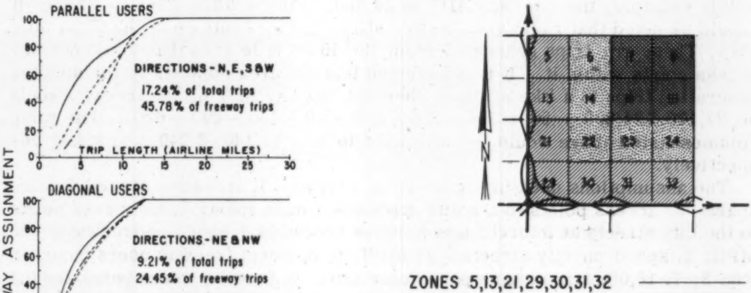

FIG. 4.—FREEWAY USERS ASSIGNMENT NORTHEAST QUADRANT 16 SQ MILE AREA

in time and distance between the city street route and the freeway route and using the family of curves shown in Fig. 3, the percentage assignment to the freeway was determined. For each mile distance the average percent assigned was computed from the individual assignments to zones included in a 45° sector of a mile ring. These points were plotted to produce the percent freeway.

Note that only one set of curves in Fig. 4 was found necessary for parallel assignment while two were needed for diagonal assignment.

Step 8.—Using the percentage freeway assignment curves in Fig. 4, the appropriate number of freeway users for each trip length was determined and assigned to the proper access point. These one-way ADT ramp volumes are shown in Fig. 2.

Step 9.— The one-way ADT leaving the 16 sq mile area on each freeway was determined by summing the volumes shown at successive access points. Thus, in this example, the one-way ADT is 24,880. (160 + 9820 + 7660 + 7240). It should be noted that ramp volumes are shown in Fig. 2 outside of the 16 sq mile area. These are trips generated from the 16 sq mile area that would not use access points within it. If it is assumed that the area adjacent to the study is generating trips in a like manner, then the oneway ADT on the freeway would be 27,120 (24,800 + 50 + 160 + 290 + 620 + 50 + 160 + 290 + 620). The ramp volumes listed above would be increased to 260, 10,140, 8,240, and 8,480, respectively.

The assumptions used in Fig. 2 are as follows: 1. freeways spaced 4 miles apart; 2. access points to the city streets - 1 mile apart; 3. no access points to the city streets at interchanges between freeways 4. speed on freeways - 52 MPH; 5. speed on city streets - 25 MPH; 6. per cent freeway users based on Fig. 3; 7. 18,000 trip origins per square mile; 9. freeway users penalized 0.1 mile and 0.5 min at each end of freeway trips to allow for circuitous ramp travel; and 10. equal desire for travel in all directions. The results may be tabulated as: 1. freeway and ramp one-way ADT volumes as shown; 2. parallel users - 17.24% of total trips 45.78% of freeway trips; 3. diagonal users - 20.42% of total trips 54.22% of freeway trips; 4. average trip length on freeways for trips 40 miles or less = 10.74 miles; 5. sum of average approach and departure distances for freeway users = 2.43 miles; 6. average trip length of non-users = 2.99 miles; 7. average over-all trip length = 6.86 miles 5.47 miles (airline); 8. 37.66% of all trips assigned to freeways, 58.59% of total vehicle miles on the freeways, 62.34% of all trips assigned to city streets, and 41.41% of total vehicle miles on the city streets (For trips 40 miles in length or less); 9. one-way ADT on the freeways from 16 square miles for all directions = 108,480; 10. one-way ADT on a freeway from 16 sq miles for each direction = 27,120; and 11. maximum one-way ADT on the freeway system for an area over 80 miles in length and width = 85,000.

It can be seen from these results that a 4 mile spacing of freeways would provide good traffic service for the conditions given. The ramps would be operating at about maximum capacity and the 85,000 maximum one-way ADT that could develop if the system were large enough is within the practical operating range of an 8-lane freeway. An indication that a reasonably good balance of service between the freeway system and the city street has been attained is demonstrated by the results numbered 4, 6, and 8. These results should not be construed as an argument for 4-mile spacing. They are primarily intended to demonstrate certain basic relationships affecting freeway spacing. Under actual conditions of variable population densities, land use patterns, and traffic desires, the freeway planner is more often faced with a good reason for deviating from a fixed spacing rather than adhering to it. Studies of this nature can serve an important function in the early preliminary phases of freeway system planning.

A deep appreciation by the highway planner of the complexities involved in the proper spacing of freeways in an urban area is an essential requirement for the successful prosecution of his mission to the public. Controls on freeway spacing include such physical controls as ramp location and adequate distance for traffic signing, such practical controls as availability of money for construction of the ultimate system, and such intangible controls as insuring that the planned system will enhance the growth and orderly development of the community it is designed to serve.

The highway planner must repeatedly strive for a long range view of the over-all, conflicting elements of freeway spacing or he will become hopelessly lost in a morass of technical detail that will only serve to complicate his problems. Freeways can no longer be considered on the basis of individual merit, but must be planned as a part of a system that will provide the best over-all traffic service to an area, as well as the greatest amount of community benefits.

ACKNOWLEDGMENT

The analysis of freeway access spacings, being done by the California Division of Highways, is under the direction of Gibson W. Fairman, M. ASCE.

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ARKANSAS RIVER PLAN

By William Whipple,[1] F. ASCE

SYNOPSIS

Development of the Arkansas River for navigation and other purposes poses especially difficult problems on account of the quantity of sediment carried. A revised concept and plan based on relationships between slopes, depths, and widths of contracted channels results in the elimination of three dams and savings of $31,000,000.

GENERAL

The development of the Arkansas River for navigation purposes from the Mississippi River some 500 miles upstream to the vicinity of Tulsa, Okla., is the most recent of the great new inland navigation systems to be initiated. The entire system is estimated to cost $1,201,000,000, including flood control and power features. It is estimated that this system will eventually carry 13,000,000 tons of waterways traffic a year where now (1960) none exists. The Arkansas River is unique in the difficulties it presents. Although it carries a heavy load of sediment, its discharge and slope are not suitable for open channel navigation such as is under construction on the Missouri River. Locks and dams must be utilized to obtain navigable depths in the Arkansas River.

Large reservoirs now under construction will retain a large part of the sediment load, and on the main stem a combination of locks and dams and channel stabilization works will be needed. The final plan has been outlined on the basis of computed relationships between the energy slopes and depths, and the width of contracted channels. This plan represents a new concept of river develop-

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ment. The result has been the elimination of at least three dams and a net savings of $31,000,000 over combinations previously contemplated.

THE OVER-ALL PLAN

The Main Control Plan of the Arkansas River will have as its principal benefit the extension of inland navigation. In addition, however, full advantage is taken of other objectives. Three main reservoirs in Oklahoma will largely regulate the Upper Arkansas and its principal tributaries, providing flood control, hydroelectric power, and regulation of flows for lockages. An essential function of these reservoirs is to retain the sediment which originates above these points. The two largest of the navigable main stem dams have sufficient head to generate hydroelectric power advantageously and will withhold further amounts of sediment. The principal characteristics of these larger projects are indicated in Table 1.

TABLE 1.—CHARACTERISTICS OF LARGER PROJECTS

Project (1)	Status (2)	Power Installation (3)	Dead Stor. & Sediment Reserve (AC FT) (4)	F. C. & Other Usable Storage (AC FT) (5)	Cost Estimate (6)
Keystone	Under Constr.	None[a]	415,000	1,464,000	$111,000,000
Oolagah	Under Constr.	None[a]	65,000	1,454,000	39,200,000
Eufaula	Under Constr.	90,000 KW	897,000	2,951,000	141,000,000
Short Mountain	Not started	110,000 KW	380,000	120,000	101,000,000
Dardanelle	Under Constr.	124,000 KW	430,000	70,000	94,600,000

[a] Power authorized, not to be initially installed.

PROBLEM AREA

The part of the plan with which this paper is primarily concerned is the river downstream from Dardanelle to the point known as Arkansas Post, where the navigation channel will leave the Arkansas River to make its exit to the Mississippi through the relatively favorable channel of the adjacent White River. This section is shown in Fig. 1.

This section of the Arkansas River will have about 190 miles of length along its improved channel. This is 27 miles less than its original length. There are no major tributaries along this reach and the minor tributaries have very little sediment content. Slope averages 0.8 ft per mile, and the variations in slope are local in character. Flood flows also are remarkably uniform, with the observed peak discharges somewhat less downstream. The river is strictly alluvial, flowing over beds of its own sediments.

The sediment content, averaging 100,000,000 tons annually, will be drastically reduced by the reservoirs. The average annual sand load of 30,000,000 tons will be reduced to an average of about 2,000,000 tons annually at Dardanelle upon closure of the Eufaula and Dardanelle Reservoirs in 1964. A quite complete channel stabilization and contraction program will be constructed, using revetments, cut-offs, and pile and rock dikes. Physically this work closely resembles similar work on the Missouri River, with sinusoidal trace and generally parallel opposite banks.

AUTHORIZED LOCKS AND DAMS

The original plan, as revised and brought up to date periodically, contemplated a series of locks and dams superimposed over the rectified channel and interferring with it as little as possible. After earlier consideration of larger numbers of dams, eleven were planned in this portion of the river. Further savings in construction costs could, of course, have been obtained by further reducing the number of dams, but the reduction was limited by the practicable height to which pools could be raised without adversely affecting drainage and inundating or waterlogging too much agricultural land. It was realized that the channel bed would ultimately be lowered by degradation processes, but these processes are relatively slow and were not relied on to increase navigable depths.

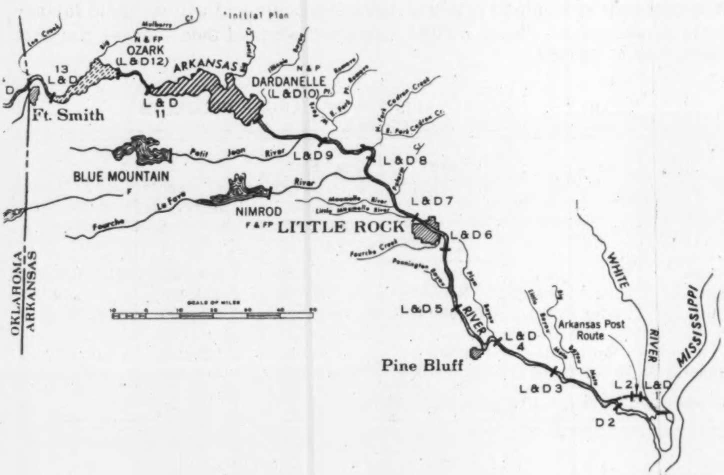
*Initial Plan
Lee Creek
Mulberry Cr
Big
N & FP
OZARK
(L & D12)
13
L & D
D
Ft. Smith
L & D
11
ARKANSAS
N & P
DARDANELLE
(L & D10)
Petit Jean River
L&D9
L&D8
BLUE MOUNTAIN
F
River
Fourche La Fave
NIMROD
Maumelle River
Little Maumelle River
L&D7
F & FP LITTLE ROCK
L&D6
Fourche Creek
Pennington Bayou
L&D5
RIVER
L&D
4
Pine Bluff
Arkansas Post
Route
L&D3
L2
L&D
1
D2
WHITE
RIVER
MISSISSIPPI
OKLAHOMA
ARKANSAS
SCALE OF MILES
10
0
10
20
30
40
50

FIG. 1.—PROBLEM AREA OF ARKANSAS RIVER PLAN

These studies were adequate for initial purposes; however, after construction funds were appropriated for upstream reservoirs and, as more planning funds became available, an intensive system study was carried on to develop the optimum system concept of structures and basis for design. Although none of the structures involved seemed to present any particular difficulty in themselves, it was apparent that the sediment problem would profoundly affect certain design considerations. In particular, it appeared that by taking full advantage of sediment characteristics of the river, it might be possible to eliminate more of the dams, in spite of practical limitations on pool elevation.

PROGRAM OF INVESTIGATIONS

A general program of sediment observations had been conducted over a period of years so that by mid-1958, when substantial funds for general system planning were appropriated, a great deal of data were already available. Also many preliminary studies had been made. To assure an adequate scientific basis for final studies, an Arkansas River Sediment Board was organized, consisting of Messrs. L. G. Straub, H. A. Einstein, and D. C. Bondurant, to review work that had been done by district staffs and to outline further studies, data-gathering, and tests required. In 1959 a large scale movable bed model was constructed at the United States Army, Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss., and tests carried on to determine sediment carrying characteristics of a regulated channel under various conditions of contraction, and with reduced slopes. Intensive analytical and design studies were also carried on, particularly by the Little Rock District, Corps of Engineers.

BASIC PRINCIPLE

Considering a system of low-head locks and dams forming navigable pools, if the upper end of a pool is excavated deeper by dredging, but also contracted an appropriate amount, it will maintain the greater depths, with a somewhat flatter water surface slope than unaltered sections above and below, but with the same total sediment carrying capacity. By this means a dam of a given elevation may extend reliable navigable depths miles further upstream and fewer dams of limited height will extend navigation over a given reach. By August, 1958, investigations were begun into the realization of this possibility. The problem was to find ways to apply such a concept quantitatively with sufficient reliability that many million dollars could be staked upon the outcome.

SEDIMENT TRANSPORT AND DESIGN

Sediment transport is a field governed by very complex laws, which are imperfectly understood. Available literature presents only general guides that might apply to this case. Fortunately, the range of conditions to be covered for the purposes of this analysis is quite limited. Absolute results are unimportant; it is only the relative sediment carrying capacity that is involved, as between consecutive reaches, with differences of slope not over 20%, coupled with differences of channel width not over 30%. As between two adjacent sections, most other characteristics will either be identical or can be controlled by design.

From a study of all references and by checking with observed results on the principal tributaries, it was decided that the approach would be concentrated on the bed material load, or the load of sands and gravels characteristic of the bed. Secondly, it was decided that the amount of bed material load could be taken as represented by the relationship:

$$\frac{Qs}{b} = K\,(D\,S)^{X} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (1)$$

in which Qs denotes the quantity of bed material load, b is the width of channel, D refers to mean depth, S is the slope, and X is an exponent. In theory, a value of 3, was determined empirically. This approach is similar in principle to the approach by H. Rouse.[2]

It was found that within the range of larger flows, the relationship of Eq. 1 on the Arkansas showed value of X generally between 2.5 and 3.0, for example, 2.8 at Little Rock. However, values were taken from experience curves, such as Fig. 2, applicable to the reach in question.

As a check a comparison was made with available "regime" theory. The origin of this theory is entirely different from that of other sediment transportation doctrine. However, since such theory is expressly concerned with design of canals of different characteristics that are designed to transport the same proportion of sediment from end to end of a network, the problem is partly analagous to what is being attempted on the Arkansas River. Direct application of equations could not be made, since regime canals have relationships between depth and width governed by rules. An analysis in terms of the D S relationship was made of a computed regime system,[3] on the assumption that sediment discharge was intended to be proportionate to water discharge in the main line and all branches. It was found that design discharge in channels of various capacities cited varied directly with $(D\,S)^3$. Therefore, it would appear that regime theory is closely consistent with the basic criterion of sediment discharge adopted for the Arkansas River.

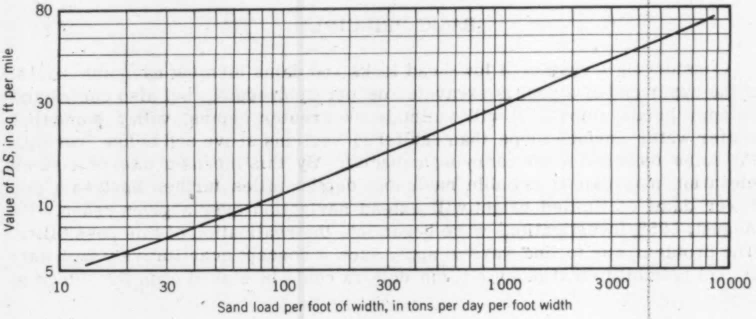

FIG. 2

PROCEDURES OF CHANNEL DESIGN

Design was initiated on the basis of a flow of 130,000 cfs, which was considered typical of the higher flows at which most of the sediment will move. With a normal contracted channel at a slope of 0.8 ft per mile this discharge will move approximately 221,000 tons of sand per day. With a given total sediment load and any other width of the channel, the corresponding sediment transport capacity per foot of width can be computed. From Fig. 2, the value of D S required to move that load is obtained. Average depths and slopes are then readily computed using normal hydraulic relationships. Navigable depths to be expected from the characteristic flow conditions in sinusoidal regulated channels were derived from mean depths on computed channels by empirical relationships.

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[2] "Engineering Hydraulics," by Hunter Rouse, Fig. 15, p. 797.

[3] "Regime Behaviour of Channels and Rivers," by T. Blench, Butterworths Scientific Publications, London, 1957, Table 3.1, p. 34.

Actual computations were somewhat more complicated than previously indicated. The characteristic trace and regulated channel actually exist only through portions of the 190 miles of river in question. Therefore, the anticipated conditions of the uniformly regulated channel had to be computed for each pool prior to computation of conditions resulting from additional contraction and excavation. Fortunately, only relative effects are important. If the total sediment carrying capacity of the more contracted sections is equal to that of the less contracted sections, actual absolute values of sediment carrying capacity are irrelevant, so long as they exceed the load delivered at the head of the system. Although the previous explanation is given for a single discharge of 130,000 cfs, analyses for a range of discharges, up to 440,000 cfs, were considered for each reach.

SCOUR AND EQUILIBRIUM

The channels thus designed are not equilibrium channels. In fact, aggregate future bed scour can be approximately computed. Starting with greatly reduced sediment below Dardanelle, the river is expected to "pick up", substantially, its full sediment load near Little Rock, Ark., at the start of navigation. This would amount to some 28,000,000 tons annually, if pools were not held up during times of low and medium flow.

No formal computation has been made, but it is apparent that average bed scour on the order of 20,000,000 tons annually is to be expected for a number of years. Initially, this will be most intense at a short distance below Dardanelle and of little effect as far downstream as Little Rock. More severe scour will occur progressively downstream, because natural armoring processes arrest the scour in upstream reaches. As degradation progresses, finer particles of bed material are removed first, leaving a higher proportion of coarser particles exposed. This process continues until the coarser bed material, being more resistant to scour, allows no more sediment to be removed than the amount coming down from upstream. At this equilibrium condition the exposed surface of the bed is characteristically covered or armored with a layer of material coarser than the average of underlying sediments.

Design of structures will have to contemplate severe degradation even below Pine Bluff, Ark., although a good many years will elapse first. Banks must of course be adequately protected, allowing for future degradation which would undermine shallow revetments. Dredging is necessary to provide adequate depths for the initiation of navigation, since degradation might otherwise take many years to reach the same depths.

Only through the armoring of the bottom will the degradation process be halted. The slope itself would never be flat enough for equilibrium otherwise. The reduction of D S values corresponding to reduced sediment availability below Dardanelle will be approximately 50%. However, a reduction of slope even of 50% would not be sufficient to accomplish this reduction, on account of increased depths at that slope for a given discharge. Of course a reduction of slope as great as 50%(at high stages) cannot occur over a great length of river. Past experience elsewhere and studies of gravel occurrence in the Arkansas indicate that the equilibrium reached will be on the basis of a coarsened bed material rather than a slope proportioned to the reduced sediment load. For example, the closing off of practically all sediment flows on the Red River at Denison Dam would theoretically entail a greatly reduced equilibrium slope for many miles below. Actually degradation proceeded fairly rapidly until water levels at normal stages had been reduced at about 6 ft. The control is a gravel bar a few miles downstream. A large flood was passed in 1957 with no further degradation.

HEAD ON DAMS

One of the corollaries of the new concept is that the navigation dams must be designed to withstand a head of several feet at high stages. Originally, it had been expected to design them like the Mississippi River dams above St. Louis, which pass flood flows with only a swell head of 1 ft. However, with a given degree of roughness, and a continuing process of degradation, there is no way in which the fall of the river can be compensated for other than by increasing head loss at the dams themselves. The planned excavation below each dam will expedite the development of this head loss, despite the accompanying contraction. However, design would have to provide for it in any event.

RANGE OF RESULTS

Results of certain preliminary computations shown in Table 2 are of interest as illustrating possible alternatives. In Table 2, a comparison of Case I and IA corresponds approximately to a comparison between adjacent sections of different degrees of contraction as outlined previously. This shows that by dredging 3,600,000 cu yd and providing necessary contraction works to contract the upper section to 1,000 ft, the navigation pool of a given dam could be extended 3.7 miles upstream. However, further consideration of cases IIA and IIIA shows that the pool could be extended much further upstream, as much as 11.5 miles, by more extensive dredging, altering the basic slope of the lower section of the pool as well as the upper, more contracted section. However, for these latter cases the head on the dam itself at high stages will be increased to as much as 4.35 ft, which is greater than desirable. Moreover, the amount of dredging involved, over 19,000,000 yd per pool, is very great. Apparently, the most economic solution in this series of approximations is Solution IIIB, which resembles IIIA except that the water surface upstream from the dam is allowed to be raised an extra 0.7 ft with a consequent raising of the design bottom elevation at points above. This solution produces 6.9 miles of extra channel, and requires only 6,800,000 yd of dredging. The head on the dam is slightly less than 3 ft, but it must be noted that water surface above the dam is raised 1.5 ft. This characteristic might be unacceptable in many locations on account of flood heights.

Such results are illustrative of the range of solutions possible within which an economic design can be worked out, satisfying actual conditions at the loca-

TABLE 2.—NAVIGATION CHANNEL STUDIES[a,b,c]

Plan	Normal depth	Bed material sediment load
I	$y_o = 26.0$	250,000 Tons/Day
II	$y_o = 26.4$	225,000 Tons/Day
III	$y_o = 26.85$	200,000 Tons/Day

Case	Basic slope (ft/mi)	Excavation (width-feet)		Contracted width (feet)		W. S. drop at dam (feet)[a]		Total	Maximum excavation (feet)	Pool length (mile)	Excavation (cu yd)
		Upper reach	Lower reach	Upper reach	Lower reach	U/S	D/S				
I	0.80	None	None	None	None	0	0	0	0	20.00	0
II	0.75	1,370	1,370	None	None	+0.39	-0.67	1.06	1.06	21.33	1,326,500
III	0.70	1,370	1,370	None	None	+0.85	-1.44	2.29	2.29	22.86	3,072,000
I-A	0.80	600	None	1,000	None	+0.00	-0.47	0.47	3.00	23.75	3,600,000
II-A	0.75	600	1,370	1,000	None	+0.39	-1.73	2.12	5.7	27.13	9,942,000
III-A	0.70	600	1,370	1,000	None	+0.85	-3.50	4.35	9.2	31.56	19,508,500
III-B	0.70	600	1,370	1,150	None	+1.55	-1.44	2.99	5.5	26.93	6,668,000

[a] Q = 130,000 cfs n = 0.0292.

[b] Normal pool assumed 28 ft above original bottom elevation at dam site

[c] Minimum depth in 600-ft navigation channel = 12 ft

[d] W. S. U/S from dam raised 0.7 ft above y_o = 26.85

tion in question. Final design studies of each structure will be required before such determination.

THE GENERAL PLAN

Meanwhile, enough has been determined to enable an approximate cost estimate to be made of seven and eight-dam systems, to compare with an eleven-dam system for the reach in question. Actually the seven-dam system proved to be the cheapest in direct total cost. However, it did not provide fully adequate navigation conditions for port development at either Pine Bluff or Little Rock; it adversely affected more bottom lands. The economic effect of this loss was not fully indicated by the cost estimates and it greatly limited the flexibility of future detailed planning of individual pools and structures. Summarizing results obtained, the eleven and eight-dam systems may be compared as indicated in Table 3.

So far as operation and maintenance is concerned, there does not seem to be any pronounced difference between the two schemes. Obviously cost of operation and maintenance of the locks and dams will be proportionately less for the eight-dam plan and dike maintenance costs will be somewhat higher. It was, at first, thought that dredging maintenance costs would be greater for the eight-dam scheme, but this will not necessarily be the case. It seems likely that crossing depths will be about as predictable for the narrower channels as for the wider. The narrower deeper channels will have the same critical areas of probable shoaling as the wider shallower channels, namely, the first

TABLE 3.—COMPARISON OF ELEVEN AND EIGHT-DAM SYSTEM

	Eleven-Dam System	Eight-Dam System
Lift of dams (low flow)	10 - 14 ft	16 - 21 ft
Construction, cost, locks, and dams	$258,000,000	$197,550,000
Cost of land taken or damaged	10,050,000	8,890,000
Acres of land taken or damaged	16,180	10,950
Additional dikes, cost	$ 3,450,000	$ 10,850,000
Dredging, cubic yards	17,150,000	46,050,000
Dredging, cost	4,100,000	11,010,000
Total cost	$275,600,000	$228,300,000
Indicated Savings		$ 47,300,000

two or three crossings downstream from each dam. Under the new scheme there will be three less of such vulnerable reaches. While this study was underway, an interim decision, affecting other sections of the river, had eliminated one dam from the eleven-dam system and reduced the indicated cost by $16,000,000 on that account. Therefore, a more conservative evaluation of the savings is $31,000,000.

As far as serving its primary purpose is concerned, the eight-dam scheme has the major advantage of reducing the time of all navigation tows by eliminating three lockages. Altogether it appears as the best and soundest scheme, and it has been adopted as the basis of further detailed planning of structures concerned.

AUXILIARY STRUCTURES

The preceding analysis has rather complex implications when considered in terms of actual structures. The contraction works so far constructed on the Arkansas River are quite similar to those on the Missouri River, although the structures are further apart, leaving an open channel of 1,200 ft to 1,300 ft. Such construction results in a series of relatively stable deep pools along the concave sides of bends, joined by intermediate shallower crossings. Fig. 3 illustrates contraction works in a reach of relatively mild curvature. This reach constituted the upper third of the movable bed model.

Principal problems in planning increased contraction in given reaches concern: (a)the possibility of changed roughness coefficients when additional contraction is introduced; (b) the necessity of avoiding appreciable increases in flood heights; and (c) the feasibility of contraction effective at higher discharges, at which times there is considerable overbank flow. In addition it is necessary to consider the possible use of special structures to assure greater depths at crossings than would be maintained by normal designs.

If it can be considered that over-all roughness remains unchanged, it appears from theoretical considerations that, for a contracted channel, a high-water flow line of somewhat lesser slope than that of the normal channel will be sufficient to carry the sediment load.

This is based on the Manning Formula

$$\frac{Q}{b} = f \left(D^{5/3} S^{1/2}\right) \ldots\ldots\ldots\ldots\ldots \quad (2a)$$

or

$$\frac{Qs}{b} = f\,(D\,S)^{2.8} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2b)$$

Dividing Eq. 2(b) by 2(a) leaves

$$\frac{Qs}{Q} = \frac{D^{2.8}\,S^{2.8}}{D^{1.7}\,S^{0.5}}$$

$$= D^{1.1}\,S^{2.3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

The condition of continuity in unit sediment load is

$$D_1^{1.1}\,S_1^{2.3} = D_2^{1.1}\,S_2^{2.3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

Provided values of n are unchanged, which is implicitly assumed in the preceding, any increase in D consistent with continuity must therefore be accompanied by a reduction in slope and a consequent lowering in the flood profile.

Therefore, in cases such as previously described, contraction of overbank flows as well as channel flows can be incorporated in the program without increasing flood heights. Such overbank contraction will supplement contraction measures by means of dikes and revetments, which are most effective in regulating the lower flows. Disposition of excavated spoil so as to channel overbank flows, encouragement of willow growth, and construction of secondary works such as Kellner Jetties in chutes and on bars not exposed to direct deepwater attack will all be considered as secondary means of contraction economical in themselves, to supplement contraction of the main channel by standard dikes and revetments. Such overbank contraction will help preserve the relative sediment-carrying capacity of contracted sections at high discharges. This is of particular importance because of the enormous loads of heavy sediments carried at such times.

The maintenance of unchanged over-all roughness coefficients in the more contracted sections involves mainly considerations of form roughness. The comparison with the natural stream need not be made, since practically all reaches downstream from Dardanelle will be contracted. With the extra contraction, the greater depth and velocity on concave sides of bends will have a tendency to increase the mean form roughness on account of obstruction by contraction works, particularly if bends are sharp and of great curvature. Obviously the adjustment of planimetric layout by making cutoffs of such bends in the upper reaches of pools may be desirable where this is feasible.

A less obvious but more widely applicable means of increasing sediment-carrying capacity is to prevent the turbulence that is created by exposed ends of spur dikes on concave sides of bends, such as those indicated as "A" on Fig. 3. Such construction has long been known to create eddying, and, in fact, such construction often partially fails due to direct attack in deep water, whereas spur dikes on concave sides of bends readily silt up. The model showed a striking improvement in over-all sediment carrying capacity when such exposed ends of spur dikes were joined by longitudinal rock dikes, with the appropriate fill behind them. This result suggests that future contracted sec-

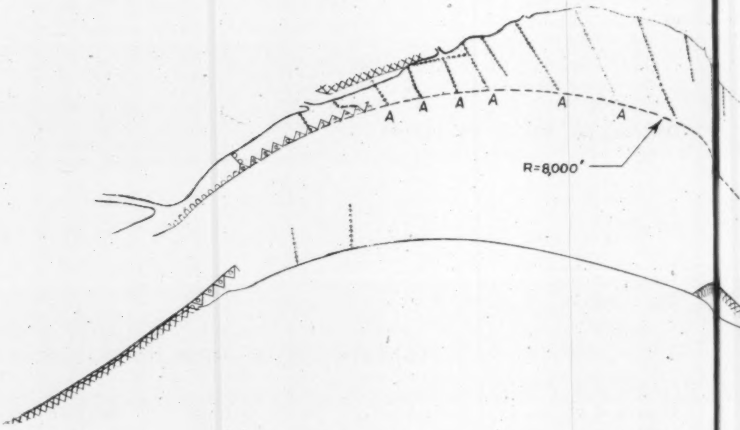

FIG. 3.—CONTRACTION WORKS IN A

tions in the upper ends of pools may profitably incorporate longitudinal rock dikes. This action would reduce the degree of contraction otherwise required, by reducing form roughness.

The third category of special contraction works which requires further consideration is that of low-level crossing groins. Such groins are not generally used, but in one form were suggested[4] by S. Leliavsky for the purpose of increasing reliable depths obtainable at a given crossing. Leliavsky proposed a single low groin, which would, in effect, continue the curve of a concave bank well into the crossing below, thus forming a low training wall for the downstream limit of the crossing.

Such groins were constructed in the open channel model, as shown on Fig. 4, together with low spur groins delineating the opposite side of the crossing channel. Improved channel depths resulted, but an unacceptable degree of turbulence was created at high stages, and shoaling was created in the channel above the crossing.

As a second test, crossing groins were installed based on a principle suggested by the author. It has been observed that following floods, the deepest paths of crossings are relatively far downstream, at a slight angle to the main axis of the channel, and that in periods of low water the crossings tend to widen and to move irregularly and unreliably upstream. Therefore, the groins were laid out parallel to the high water flow, located so as to limit the extension of the crossing upstream, as shown in Fig. 4. These crossing groins did not perceptibly interfere with flow of the current in the model, or apparently suffer

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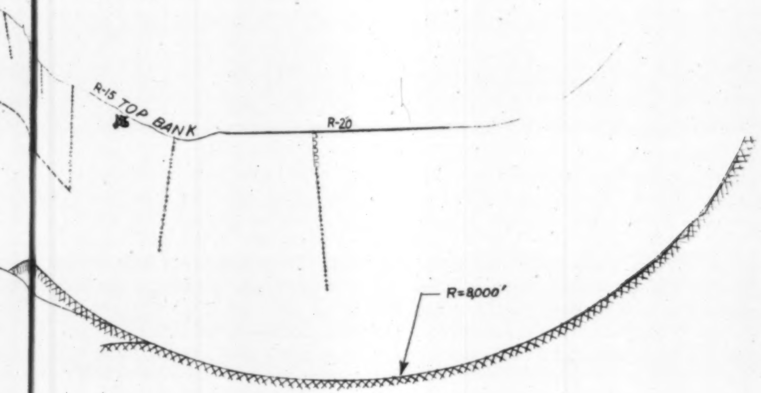
[4] "An Introduction to Fluvial Hydraulics," by Serge Leliavsky, Fig. 40, p. 104.



REACH OF RELATIVELY MILD CURVATURE

any direct attack at high stages. They materially improved crossing depths. Considerable fill resulted between the groins and the bank after passage of flood stages.

Consideration of these tests indicated possible economy of such groins, or some other generally similar structure, for application particularly to those crossings where designed depths cannot otherwise be maintained without maintenance dredging. There will be time for such auxiliary structures to be more fully developed and tested before the first navigation dams come into permanent use.

CONCLUSIONS

The program for extending navigation up the Arkansas (and Verdigris) Rivers to the vicinity of Tulsa, Okla., is one of the largest multiple-purpose river improvement programs ever adopted. It involves novel and complex problems from an engineering viewpoint. The initiation of construction of three of the major structures in 1955, at a time when preconstruction planning of the system as a whole was not very far advanced, represented great reliance upon the accuracy and correctness of the highly generalized pre-authorization studies and certain very preliminary approaches.

Fortunately, these conclusions require no major change in the scope and concept of the projects already under construction, except for a considerable reduction in the tailwater elevation of Dardanelle Dam, which was worked out on an interim basis prior to completion of more general studies involving the same principles. The writer and many others in the Corps of Engineers feel a profound relief that this great project, unprecedented in character, and in-

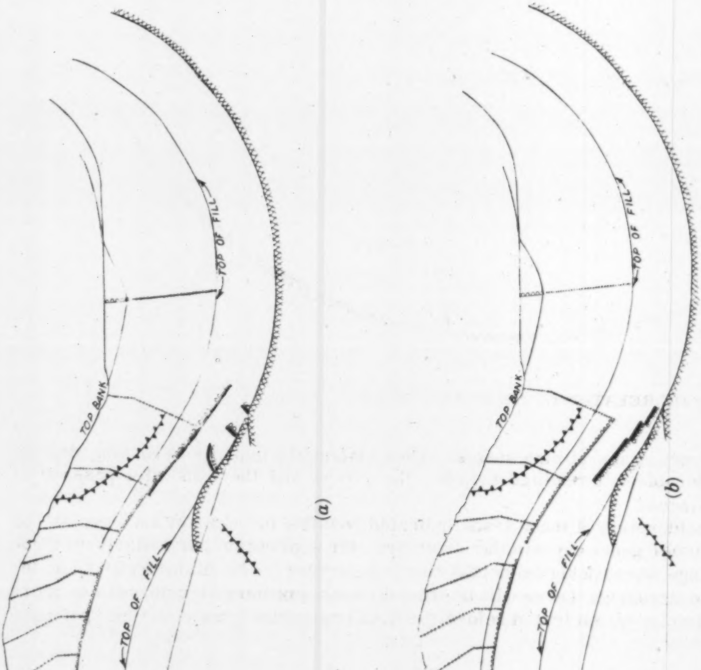

LEGEND
SPUR LEVEE
TOP BANK (APPROXIMATE)
PILE DIKE WITH STONE FILL
STONE-FILL DIKE OR REVETMENT
PILE REVETMENT WITH STONE FILL
STANDARD REVETMENT
SUBMERGED GROIN
TOP BANK
TOP OF FILL
TOP OF FILL
(a)
TOP BANK
TOP OF FILL
TOP OF FILL
(b)

FIG. 4.—PLACING OF LOW-LEVEL CROSSING GROINS

itiated prior to the solution of all the recognized problems, is adhering to the early estimates and even indicating the possibility of savings, based upon current price levels.

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ELECTRONIC COMPUTER IN HIGHWAY ENGINEERING

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SYNOPSIS

This paper describes electronic computers and their functional components as well as the way in which they operate, the procedure for using them in the solution of engineering problems, and applications to highway engineering problems.

INTRODUCTION

In 1955, the electronic computer was virtually unknown in the highway and highway bridge engineering fields. Today (1960), all but a few of the state highway departments are using electronic computers in their day-to-day engineering work. They have installed their own computers, trained their engineers to use them, and have established computer staffs to operate them. Consulting firms engaged in highway and bridge engineering also have been quick to recognize the advantages inherent in electronic computation and more of them are turning to this new device. Electronic computation is now well established and is fast becoming standard practice in highway and bridge engineering organizations.

In view of the capabilities of the electronic computer, it is not surprising that there has been this rapid and broad acceptance of it. A large part of engineers' time is consumed in the processing of field and laboratory data and in

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performing computations involved in research and development, planning, location, design, construction, and estimation. Much of this computation is routine and repetitive and, in itself, adds nothing to the engineer's proficiency. Also, with few exceptions, the time required to complete an analysis or to advance an approved project to the construction stage is determined by the time required to process data and to perform the necessary computations. At times, the sheer magnitude of the computing job forces the engineer to limit his investigation and base his findings on data and analyses that are not as complete as they could be. In other instances, needed engineering and economic studies are deferred or not undertaken at all because of the seemingly prohibitive volume of data processing and computation involved.

In the electronic computer, the engineer has found the remedy for these difficulties, a device which can process data at amazing speeds and perform literally thousands of arithmetic operations per minute with perfect accuracy and without his supervision.

TYPES OF COMPUTERS

All computing machines or devices, whether or not they operate electronically, fall into two general categories of analogue and digital. The basic difference between them is that the analogue computer handles data in the form of measurements of physical variables while the digital computer uses numbers in digital form. Each has its field of application and a substantial record of achievement, both of which have broadened substantially with the advent of the electronic computer.

Analogue Computers.—In the analogue computer, numbers or quantities are represented analogously by lengths along a scale, shaft rotations, electric currents or voltages, and so on. The slide-rule is an example of an analogue computer in which numbers are represented by lengths along a scale. An example in which shaft rotations are used is found in the automobile speedometer. Rotations of the drive shaft are converted to a visual indication of the speed of the vehicle. Other examples of non-electronic analogue computers include planimeters, nomograms, household watt-hour meters, thermostats and many other devices in common use.

Most electronic analogue computers represent numbers or quantities by direct current voltages. The electronic circuitry of the computer is arranged to simulate mathematically the problem to be solved or the physical system under study. The input voltages, representing numerical values for the constants and variables involved, are adjusted or modified by means of potentiometers to correspond to the varying conditions to be analyzed. Answers are read on mechanical indicating dials, displayed on cathode-ray tubes or drawn by pen recorders. While electronic analogue computers may be and sometimes are constructed as general purpose computers, more frequently they are special purpose devices, that is, they are designed for the solution of a particular problem. They are particularly valuable in solving dynamic problems in which the input values vary continuously with respect to time. More applications are being developed in process control and industrial automation, as well as in scientific and engineering problems. However, because of the limitations involved in physical measurement, it is difficult to attain a precision greater than three or possibly four significant figures. For this reason, the electronic analogue computer has not been considered as useful for highway and bridge engineering problems as its digital counterpart.

Digital Computers.—In the digital computer, practically any degree of precision consistent with the precision of the source data can be attained simply by specifying the number of decimal places to be used.

The general purpose electronic digital computer is the type of computer which is proving to be so valuable in highway and bridge engineering and it is the one with which the remainder of this paper is concerned. As stated previously, in a digital computer, numbers are represented in terms of their digits. The venerable abacus, still rendering good service in parts of the world. adding machines, desk calculators and bookkeeping machines are examples of nonelectronic digital computers.

The first electronic digital computer was developed at the Moore School of Electrical Engineering of the University of Pennsylvania for the Ballistic Research Laboratories of the Army Ordnance Department. This machine, called the ENIAC (Electronic Numerical Integrator and Computer), was completed and placed in service at the Aberdeen Proving Ground in January 1947. This marked the beginning of the widespread use of electronic digital computers. Subsequent development was intensive and rapid with substantial advances in speed, capacity, reliability and versatility. The first machine to be mass produced was the UNIVAC (Universal Automatic Computer). This computer was placed on the market in 1951. Others followed and today many makes and models are available, mostly of the general purpose type.

Not all of these computers are the large, costly machines commonly referred to in the popular press as "giant electronic brains." Rather, they cover a variety of sizes, ranging from the North American Aviation Company's fully transistorized RECOMP, which occupies approximately 5 cu ft of space, to the large systems such as the International Business Machines (IBM) Corporation's Model 704, which requires approximately 3,000 sq ft of floor space. Between these extremes, there are a number of others that approximate, in size, an office desk or file cabinet.

However, regardless of size, they all do essentially the same things; they add, subtract, multiply and divide. In this respect, they are similar to the desk calculator except, of course, that they use electronic circuitry. Also, they differ from the desk calculator in that they can store large quantities of information, they can compute at tremendous speeds, they can perform automatically long series of arithmetic operations in predetermined order without human intervention, and they can make simple logical decisions by comparing one number with another and following either of two courses of action depending on which number is the larger.

The high computing speeds are the result of using electronic circuitry to perform arithmetic operations. No wheels, levers, cams or other mechanical devices are used and, consequently, the limitations inherent in mechanical systems are not involved.

Of even greater importance than speed is the computer's ability to perform long sequences of arithmetic operations automatically. This is done through a process called "programming." Briefly, programming consists of reducing the problem to be solved to elementary arithmetic operations and then preparing a series of instructions for the computer to follow in performing these operations in proper order. The entire set of instructions for a particular type of problem is the computer program, or simply the "program" for that problem.

Once completed, it may be used over and over again, whenever that type of problem is to be solved. Most modern computers are referred to as "stored program machines." This means simply that the program being used is stored in the computer along with the problem data.

The decision-making feature is valuable in making provision for the automatic following of one of a number of alternate courses depending on given conditions or on conditions that appear in intermediate results as the computation progresses. This is done, for example, in repeating segments of the program a prescribed number of times, in determining degree of convergence in function evaluation, in table look-up or search operations, and in a number of other situations encountered in the automatic solution of a problem.

COMPONENT PARTS

While the detailed arrangement and construction of general purpose, stored program, electronic digital computers vary, the basic functional components are essentially the same. These may be represented schematically as shown in Fig. 1.

Input.—The function of the input is to place problem data and program instructions in the memory of the computer. The technical term for this operation is "reading." Both types of information are recorded in advance on punched paper tape or magnetic tape depending on the input facilities of the computer being used. Some computers will accept information only from punched cards, others only from punched paper tape or only from magnetic tape, and still others from all three media.

Problem data are recorded on punched cards by punching holes in various positions in the vertical columns of the card. Instructions are recorded similarly using a numerical code. On paper tape, both types of information are represented by code patterns of the presence or absence of holes across the width of the tape. The holes and spaces are positioned in parallel channels running the length of the tape. Depending on the code used, there may be from five to eight such channels. Information is recorded on magnetic tape in a somewhat similar fashion using magnetized spots instead of holes.

Provision is also made for entering instructions and data by means of manually operated switches on the computer itself, a numerical keyboard of a specially connected electric typewriter.

The punched cards or punched paper tape are read by metallic brushes or photoelectric cells. As the cards or tape pass through the reader, a circuit is closed each time a hole appears and a pulse of current enters the machine. For magnetic tape, an electromagnetic reading device is used.

For problems in which there is a large volume of input data, efficient operation of the computer requires that input data be read into the computer as rapidly as possible. Magnetic tape is much the fastest input medium with reading speeds up to approximately 60,000 characters per second. Where source data is on punched cards, card to tape converters can be used to transfer the data to magnetic tape. Since the converter is an independent machine, this can be done while the computer is otherwise engaged.

Memory.—The function of the memory, sometimes called the storage unit or simply the store, is to store information for use as needed in the course of the computation. In stored-program computers, program instructions as well as problem data, intermediate results and output data are stored in the memory.

Since the memory must provide for the storage of a large volume of information used in the computation, provision is made for the rapid insertion and extraction of specific items of information in and from it as needed. This is usually done by dividing the memory into segments, commonly called "cells" or "registers," each large enough to hold a number in coded form. Each of these memory registers has an identifying number called its "address," an arrangement somewhat like the numbered cubby holes for room keys and mail at a hotel desk. In this way, instructions expressed in numerical code, problem data, and the results of computation may be stored in specific, identifiable locations in the memory so that they can readily be located and read when needed without searching the entire memory.

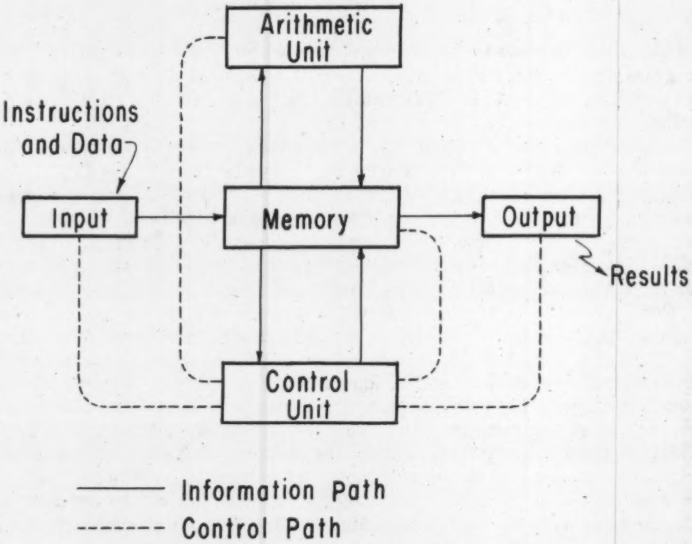

FIG. 1.—TYPICAL ARRANGEMENT OF ELECTRONIC DIGITAL COMPUTER FUNCTIONAL COMPONENTS

Memory devices are commonly classified with respect to "access time" which is the time required to locate and read an item of information stored in the memory. The relative terms "slow," "medium" and "fast" access are used.

Commonly used fast access memory devices include magnetic cores, cathode-ray tubes and sonic delay lines. The fastest of these is the magnetic core matrix which has an access time of a few millionths of a second. A magnetic core is a tiny toroid (approximately 2 mm in diameter) made of a ferromagnetic material. The cores are strung on wires in matrix form with one wire running from top to bottom through each column and a second wire running from side to side through each row. A third wire passes diagonally through each core. By sending current through a pair of wires, one horizontal and one vertical, the core located at their intersection can be magnetized. By passing current through selected pairs of wires, a number can be stored in the form of a code pattern of magnetized and non-magnetized cores or of positively and negatively magnetized cores. Many numbers can be stored in this way in a single core matrix. The total memory unit consists of a number of connected matrices. Information is read from the magnetic core memory through the diagonal wire, sensing the magnetic state of each core in the group comprising a memory register.

The cathode-ray tube memory unit consists of a bank of tubes similar to the picture tubes used in television sets but smaller. Information is stored on the faces of the tubes in the form of spots of electrostatic charge. Something over a thousand such spots can be stored on a 3-in. tube face. Information is recorded in a specified memory register on the face of a tube by directing a beam of electrons toward the spot bearing that address. In reading stored information, the electron beam is directed toward the memory register involved, producing a change in the charge stored there that is detected by a capacitive coupling to a conducting surface placed against the face of the tube on the outside. Like the magnetic core memory, the cathode-ray tube access time is a few millionths of a second.

In the sonic delay line type of memory, also called an acoustic delay line, series of electrical pulses representing, in code-pattern, the information to be stored, are transformed to a train of sound waves in a physical medium such as a tube of mercury. At the receiving end of the line, the sound waves are converted back to electrical pulses, amplified, reshaped and fed back into the mercury line at the other end as sound waves. This is a continuous process by means of which the information is kept circulating for use as needed. Information stored in this way is read at the output end of the line through the use of an address that specifies the time position in the wave train of the item desired. The address also identifies which of a number of lines comprising the total memory unit is to be used. This is a cyclic type of storage in which access time depends on the length of the delay line and the speed of sound in the physical medium used. While classified as a fast access memory device, access time is appreciably longer than for the non-cyclic magnetic core and cathode-ray-tube types of memory.

The most widely used type of memory device is the magnetic drum. This consists of a non-magnetic metal cylinder coated with magnetic material, usually a nickel alloy or iron oxide, which will accept and retain information in the form of magnetized spots. The magnetized spots are arranged in code patterns similar to the pattern of magnetized cores in the magnetic core memory. While the computer is in use, the drum rotates continuously at a high, constant speed. Electromagnets, called "read-write heads" are mounted on a casing surrounding the drum and positioned about a thousandth of an inch from its surface. These are ranged along the length of the drum and at intervals around its periphery so that as the drum rotates, information can be "written" in or "read" from any of the memory registers. The registers are arranged in parallel tracks or bands around the drum. Each track has at least one read-write head. Sometimes, more than one head is used on a single track to reduce access time.

When one of the electromagnets is energized by an electrical impulse, the spot on the drum surface that is under the electromagnet at that instant is mag-

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netized. If no current is flowing in the coil of the electromagnet, no magnetization takes place. In the reading process, when a magnetized spot passes under one of the electromagnets, a voltage pulse is induced in the coil; exactly the reverse of the writing process. A typical magnetic drum memory unit for an intermediate size computer may have as many as 4,000 memory registers. Like the sonic delay line, the magnetic drum provides a cyclic type of storage. Access time, which depends on the speed of drum rotation, is a few thousandths of a second, the medium speed category.

Another medium access time memory device utilizes a rotating disk instead of a cylinder. Information is stored on the surface of the disk in concentric circular channels, each channel containing a number of memory registers. The surface of the disk is coated with ferro-magnetic material and the read-write operations are similar to those used with magnetic drums.

Multiple disk units are used to provide large memory capacity. For example, the IBM RAMAC (Random Access Method of Accounting and Control) utilizes fifty disks on a vertical shaft providing a total of 50,000 memory registers, each of which is capable of holding a hundred digits and each having an identifying address. An access arm, carrying read-write heads, moves automatically up and down, and in and out, to reach any specified address. A number of these units can be integrated into a computer system to provide still greater memory capacity.

Magnetic tape has already been mentioned under "Input" and will be mentioned again under "Output." In addition to these two uses, it also serves as a memory device. While it is a fast input or output medium, it is relatively slow as a memory device. However, because large volumes of data can be stored in compact form at relatively low cost, magnetic tape is valuable as auxiliary memory, supplementing the more costly, higher speed types of main memory. Also, it provides a permanent type of storage. That is, reels of tape containing stored data can be filed away until needed.

Magnetic tape is made of non-magnetic metal or plastic with a coating of iron oxide or other material that will accept and retain spots of magnetism. The magnetized spots are arranged in code patterns as in the case of the magnetic drum. Also, electromagnet read-write heads with magnetic tape are similar to those used with the magnetic drum. Information is recorded in blocks that are usually addressable by means of a serial indexing system. The tapes, commonly from $\frac{1}{2}$ to 1 in. wide, are mounted on reels in tape handling units, a number of which can be used in combination. A reel may contain 2,400 ft or more of tape requiring several minutes to search from one end to the other.

To shorten the search time, multiple magnetic tape units have been developed. In these units, conventional tape reels are replaced by a number of shorter lengths of tape suspended loosely over guide rods in a bin. The length of each tape may be only a tenth of the length of the conventional reel tapes and the access time is correspondingly reduced. A number of multiple tape units can be wired into a computer system, giving a large capacity.

Another magnetic tape storage device, the Clevite Corporation's Tape DRUM, uses a magnetic tape 30 in. wide, draped over a rotating drum. The read-write heads are mounted in the drum and rotate with it. The capacity of the section of the tape that can be scanned at one time, called a page, is comparable to the capacity of a magnetic drum. The full length of the tape contains a number of such pages, each of which can be moved into position to be scanned by the rotating heads. Access time within any one page is comparable to that of the magnetic drum type of memory, but additional time is, of course, required to move the desired page into scanning position.

The costs of memory devices are closely related to access time so that various combinations of storage media are used to provide both fast access and large capacity without excessive cost. The current trend in the large computers is toward the use of magnetic cores for the fast access main memory supplemented by magnetic drums, tape units or multiple disk units. The magnetic drum has become practically the standard memory device for the small and many of the intermediate size computers. Some of the latter have provision for auxiliary magnetic tape storage but, in many cases, the drum constitutes the entire memory.

In reading data from the memory, the stored information is not erased. A number stored in a memory register remains there unless it is replaced by a new number or unless the memory is cleared. However, in the case of power failure, information stored in "volatile" storage media such as electrostatic tubes and sonic delay lines, will be lost. "Non-volatile" storage media such as magnetic cores, drums, disks and tapes, retain stored information in the absence of power, ready for use when power is restored.

Arithmetic Unit.—The arithmetic unit consists of electronic circuitry (electron tubes, transistors, diodes, and so on) by means of which the arithmetic operations of addition, subtraction, multiplication and division are performed. Also, it can shift numbers right or left, transfer data from one memory register to another, modify program instructions and assist in logical decision operations.

The basic element of the arithmetic unit is the "adder," an electronic circuit designed to obtain the sum of two numbers. Subtraction is performed by using the adder to add one number to the complement of the other. Multiplication is accomplished by repetitive addition and shifting, and division by repetitive subtraction and shifting.

The mode of operation may be either serial or parallel. In the serial type of operation, digits are handled one at a time in succession. In the parallel system, all digits of a number are handled simultaneously. Although it is slower, the serial system is the more common because it requires much less circuitry. Since the electronic elements used in the arithmetic unit respond to an electrical impulse in less than a millionth of a second, extremely high computing speeds are possible. In the intermediate size computers, several hundred additions can be performed in a second. The large computers operate at the rate of several thousand arithmetic operations per second.

Control Unit.—The control unit consists of electronic circuitry the function of which is to interpret program instructions and to direct the operations of the other parts of the computer in accordance with them.

As mentioned previously, in stored-program computers, program instructions in coded form are stored in the memory. In the course of the computation, these instructions are automatically sent to the control unit, one at a time, in proper sequence. Each instruction is stored temporarily in the control unit until it is executed and replaced by the next instruction. This process continues automatically until the entire program has been executed step by step, and the prescribed computations completed.

In externally programmed computers, control is provided through removable wired plugboards, punched cards or punched tape.

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Output.—The function of the output unit is to take the final results of the computation, and other information desired, from the computer and record them in usable form. The technical term for this operation is "writing." Various writing devices are used.

In some computers, an electric typewriter connected directly to the computer is used. Information to be recorded is drawn from the memory in the form of electrical pulses in code patterns. The pulses energize electromagnets which in turn actuate the keys of the typewriter. In this process, the pulse patterns are translated into letters and numbers so that the typewritten material is in readable form. Typing is done at the rate of from ten to fifteen characters per second.

To provide higher speeds, line-printers are used, particularly where the volume of output data is substantial. In these machines, an entire line of as many as a hundred and thirty characters are printed at one time. Speeds as high as 1,500 lines per minute are achieved. The lines of type are set automatically in accordance with the output pulse patterns. The line-printer can be connected directly to the computer in what is termed an "on line" operation or it can be used independently as an "off line" operation. In the latter case, which is the usual one, the output data is recorded on punched cards, punched paper tape or magnetic tape, which can be used subsequently to actuate the line-printer or a typewriter not connected with the computer to produce a printed record. The data is recorded on cards or paper tape by means of electromagnetically operated punches and on magnetic tape by means of electromagnetic recording heads similar to those used in recording data on a magnetic drum.

Output data is also recorded on cards or tape when it is to be used as input in subsequent computation or processing. Magnetic tape is the fastest medium for output as well as for input, with writing speeds of approximately 60,000 characters per second. In addition, it provides a compact and permanent means of filing data. Also, it can be used over again like the tape used in tape recorders. As mentioned in the section on input, converters are available by means of which data stored on cards can be transferred to tape. Converters are also available for the reverse process, transferring data from tape to cards.

Cathode-ray tubes are also used as output devices. Characters or traces of an electron beam displayed on the tube are photographed to provide a permanent record. This is a relatively new (1960) development which appears to have interesting possibilities for some types of engineering problems.

While there are differences in arrangement, construction, electronic elements and circuitry, the basic functional characteristics are as described. The arithmetic unit, control unit and memory, taken together, are sometimes referred to as the "central processing unit" and are frequently housed in the same cabinet in the small and intermediate size computers. Most computers permit the use of more than one type of input and output device and these usually are separate units.

OPERATING PROCEDURE

In the operation of the computer, the basic procedure is as follows. The program instructions, on tape or cards, are read by the input unit and stored in the memory. Problem data follows, both constants and values of independent variables. The first instruction is moved from the memory to the control unit where it is stored temporarily, interpreted and executed. The second instruction is moved from the memory to the control unit where it replaces the temporarily stored first instruction. It is then interpreted and executed. This continues until the entire series of instructions constituting the program has been executed and the results of the computation written out through the output unit. If, for example, the problem involved the determination of the sum of two numbers, the first instruction would specify the memory address of the first number and call for it to be moved from the memory to the arithmetic unit. The second instruction would specify the memory address of the second number and call for it to be moved from the memory to the arithmetic unit and there to be added to the number already in the arithmetic unit. The third instruction would call for the sum of the two numbers to be moved from the arithmetic unit to a memory location of specified address. The fourth instruction would call for this sum to be moved from the memory to the output unit to be written out. While this is a simple example using only four instructions, it serves to illustrate the working process. Many engineering problems require several hundred instructions but the basic process is the same.

PROGRAMMING

In its broad sense, programming covers everything that has to be done to prepare a problem for solution on the electronic computer. This includes, first of all, defining the problem clearly, deciding on a method or plan for its solution, and setting up the mathematical equations involved.

Because the computer's mathematical ability is limited to addition, subtraction, multiplication and division, the problem must be resolved into these basic arithmetic operations. If trigonometric function, square root, integrals or other terms that cannot be handled directly are involved, numerical methods must be used, which require only arithmetic operations such as finite differences, infinite series, iterative procedures, and so on.

Knowing what arithmetic operations have to be performed, the next step is to determine how they may be arranged to best advantage for solution on the particular computer to be used. For this purpose, it is helpful to construct a diagram or flow chart showing the overall procedure to be followed in solving the problem. The chart should show the major segments of computation and the sequence in which they should be undertaken. Consideration must be given at this time to the precision desired, memory requirements, coverage of special cases or alternate conditions, error checks, output format and other pertinent factors.

The next step is the preparation of the program. This involves the detailed step by step procedure for solving the problem. This step may take the form of a listing of instructions in word form, in proper sequence, to be followed by the computer in performing the arithmetic operations and logical decisions necessary in developing the solution. Alternatively, it may take the form of a detailed flow chart, an expansion of the overall flow chart previously mentioned, showing each operation and decision and their exact positions in the computational procedure. The chart should not itemize each instruction nor should it specify the manipulation of the parts of the computer itself. It is, essentially, a detailed pictorial representation of the problem-solution-logic in connected sequence. A listing of the major mathematical operations to be performed is helpful as a guide in constructing the detailed flow chart. In one form of flow charting, brief procedural statements are shown at the side of the chart correlated with the charted operations.

From the program in word form or in flow chart form, a "coded program" is prepared. This is a detailed listing of machine operations corresponding to the operations listed in the word form program or shown in the flow chart. Each machine operation is expressed in the code used by the computer on which the problem is to be solved. This process, termed "coding," is a tedious task that requires an intimate knowledge of the computer involved. It need not, however, require more than a superficial understanding of the problem to be solved or the method of solution. Because coding is so time-consuming and exacting, short cuts have been developed in which the computer itself does much of the detailed work.

The coded program, when completed, is a list or tabulation that includes, in proper sequence, all machine operations in coded form. The terms "coded instruction," "machine instruction," or "program instruction" are used in referring to a coded machine operation.

Before the program can be used, the coded instructions must be recorded on the input medium to be used (punched cards, punched paper tape or magnetic tape). This is done on manually operated card punches, tape punching typewriters or keyboard operated magnetic tape recording machines. If cards are used, the complete assembly of punched cards, in proper order, is called the "program deck." For either punched paper tape or magnetic tape, the tape on which the coded instructions are recorded is called the "program tape."

Except for testing, the program is now ready to be used. It is unusual for a coded program to be entirely free from errors when it is first compiled. Errors in logic can be made in the analysis of the problem and in the development of the program. Also, because of the great volume of detail, coding is a common source of error. All errors must, of course, be found and corrected before the coded program can be placed in production. This is done by comparing the answers obtained by manual solution of a test problem or problems with the answers produced by the computer. If they differ, a systematic check is made to locate the errors. This is usually done by first checking intermediate results to localize the errors and then checking the suspected segments step by step. The segments can be checked by means of manual controls or through the use of special testing programs in which the computer itself assists in locating the errors. This testing and correcting process is termed "debugging."

After all errors have been corrected, the program is ready for use for all problems of the type for which it was developed. For example, if the program was developed to compute latitudes, departures and error of closure for a survey traverse, it can be used for any survey traverse consistent with limitations that the program may have with respect to number of courses and unknowns. When such a problem is to be solved, the program deck or program tape is taken from the program files and placed in the input unit. The computer is started and the instructions are automatically stored in the memory. The course length, and bearings, also on cards or tape, are then entered into the computer (or they may be entered manually from a keyboard) and the computation proceeds. Answers are written out by the output unit. The program deck or tape is then returned to the program files where it remains until another traverse is to be computed. Like a phonograph record, it can be used over again whenever it is needed.

In the programming operation, the analysis of the problem, the determination of the method of solution, the setting up of the mathematical equations, the construction of the overall flow chart and the preparation of the program, either in word form or in detailed flow chart form, should be done by an engineer thoroughly familiar with the type of problem involved. This engineer should also know the capabilities and limitations of the computer being used and should have received training in computer programming. He need not know how to operate the machine. Experience has shown that much better results are obtained by providing brief intensive training in computer characteristics and programming procedures for selected engineers than by trying to teach a non-engineer, who may be a computer expert, how to solve engineering problems. The coding part of programming can be done quite satisfactorily by a non-engineer who has been trained for that kind of work. The programming operation up to the coding point requires approximately 80% to 85% of the total preparation time. Thus, coding and debugging the program require approximately 15% to 20%.

SUBROUTINES

The programming task may be lightened considerably by the use of "library subroutines" for operations and functions that occur frequently in engineering problems. These are prefabricated standard sequences of computer instructions, complete in themselves, for finding square roots, solving simultaneous equations, computing trigonometric functions, logarithms, exponential functions, and so on. Instead of writing out the detailed computer instructions each time one of these types of operations is encountered, they are written only once and then incorporated into any program as a packaged unit or "subroutine." For any specific problem, the required subroutines are stored in the computer's memory and are automatically called into use in the program as needed by specifying the address of the first step of the subroutine. Provision is also made to return to the main program automatically following the last step of the subroutine.

AUTOMATIC CODING

This same device is used in enlisting the aid of the computer itself in developing coded programs. This is done through the use of either "compiling routines" or "interpretive routines." In either case, symbols are used in the initial program that are broader in scope than machine codes and that represent operations familiar to the programmer such as multiply, compare, subtract, divide, and so on.

In the compiling process, as each such simplified instruction is read by the input mechanism, it is transformed into the sequence of instructions in machine code necessary to conduct the indicated operation. This is done by means of stored subroutines of coded instructions. The results are assembled and written out on tape or cards as a complete coded program that may then, or at some later time, be reentered into the computer, stored and executed in the usual way.

In the interpretive process, the symbols or simplified instructions are actually stored in the computer's memory. As each such instruction is reached, it brings into play a sequence of machine coded instructions stored as a subroutine corresponding to the simplified instruction involved. Each simplified instruction is translated and executed as it is reached.

The principal advantages of automatic coding is that it simplifies the programming task and reduces the possibility of errors in coding. However, machine produced programs frequently take longer to run and use more memory space than equivalent programs developed manually.

PROGRAMMING EXAMPLE

A simple example will serve to illustrate the conventional programming process. Assume that the equation $(x + y)c = Z$, in which x and y are independent variables and c is a constant, is to be solved for several hundred pairs of values of x and y to obtain the corresponding values of Z.

In word form, the instructions to the computer would be the same as would be furnished a person solving the problem manually:

1. Read a pair of values for x and y
2. Add x and y
3. Multiply this sum by c
4. Write the result as Z
5. Return to step 1 and repeat the sequence of instructions with a new pair of values for x and y

This is the program, in word form, for computing any problem of the type $(x + y)c = Z$ such as would be used in computing the area of a trapezoid, the quantity of excavation by the average end area method, etc. The program, in flow chart form, would be as shown in Fig. 2.

In order to solve the problem on an electronic computer, the program must be translated into the code which the computer has been designed to use. Each make and model of computer has its own code. For purposes of illustration, the fictitious code indicated in Table 1 is assumed.

The two-digit codes shown in Table 1 define the operations to be performed. In addition to the operation code, a coded instruction must include the memory address of the number to be operated on, and in the system illustrated herein, the address of the next instruction to be executed. Each of these addresses in this case will have four digits so that the complete coded instruction will have ten digits arranged in this pattern:

Operation Code	Data Address	Instruction Address
xx	xxxx	xxxx

Returning to the example used for illustration, the coded program can now be written. The memory address of each instruction and an alphabetic abbreviation of the operation to be performed are usually listed by the coder with the coded instructions. Memory addresses to be used for values of x, y, c, and Z are arbitrarily selected as 0051, 0052, 0053, and 0127, respectively, and for the instructions, a block of addresses beginning with 0501. Successive values of x and y, as they are read, will be stored in the same memory locations, 0051 and 0052, each new pair of values replacing the previously stored pair.

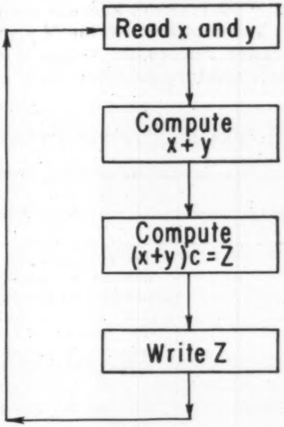
Read x and y
Compute x+y
Compute (x+y)c = Z
Write Z

FIG. 2

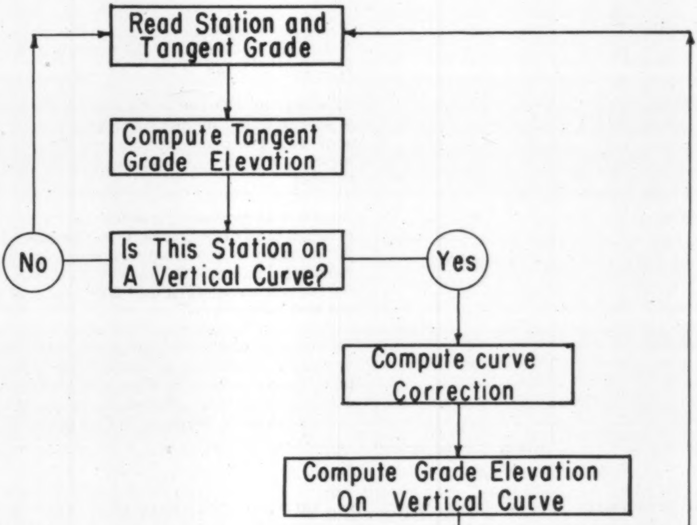
Read Station and Tangent Grade
Compute Tangent Grade Elevation
Is This Station on A Vertical Curve?
No
Yes
Compute curve Correction
Compute Grade Elevation On Vertical Curve

FIG. 3

In the read instruction, only the memory address where the value of x is to be stored need be specified. The companion value of y will automatically be read into the next memory register. Successive values of Z, as they are computed, will be stored in the same memory location, 0127, each new value replacing the previously stored value, which, by that time, will have been written out. The value of the constant c remains unchanged in memory location 0053 and, therefore, need be read only once. It can be read and stored at the time the program instructions are stored and need not be provided for in the computer instructions for performing the computation.

TABLE 1.–EXAMPLE OF COMPUTER CODE

Instruction (1)	Operation Code (2)	Description of Operation (3)
Clear and add	01	The arithmetic unit is set to zero and the number at a specified address in the memory is then placed in the arithmetic unit.
Add	02	The number at a specified address in the memory is added to whatever number is already in the arithmetic unit.
Subtract	03	The number at a specified address in the memory is subtracted from whatever number is already in the arithmetic unit.
Multiply	04	The number at a specified address in the memory is multiplied by whatever number is already in the arithmetic unit.
Divide	05	Whatever number is already in the arithmetic unit is divided by the number at a specified address in the memory.
Store	06	Whatever number is in the arithmetic unit is stored in the memory at a specified address. The previous contents of that memory register are replaced by the new number.
Read	07	Numbers recorded on punched cards, punched tape or magnetic tape are stored in the memory at specified addresses by the input mechanism. The previous contents of the memory registers used are replaced by the new numbers.
Write	08	Numbers at specified addresses in the memory are recorded on punched cards, punched tape, magnetic tape or a printed sheet by the output mechanism. The numbers are not erased from the memory in this process.

The coded program then will be:

		Coded Instruction			Explanation
0501	RD	07	0051	0502	Read x to 0051, y to 0052
0502	CA	01	0051	0503	Clear and bring x into arithmetic unit
0503	ADD	02	0052	0504	Add y to x
0504	MULT	04	0053	0505	Multiply (x + y) by c
0505	ST	06	0127	0506	Store (x + y)c = Z in 0127
0506	WR	08	0127	0501	Write Z and return to instruction 0501

The first instruction, 07 0051 0502, directs the computer to take the first number (x) punched on the problem data card and store it in memory location 0051, to store, in sequence, any additional numbers included in the punched problem data in succeeding memory locations, and then to go to memory location 0502 for the next instruction. The second instruction, 01 0051 0503, directs the computer to set the arithmetic unit to zero, then to bring into the arithmetic unit the number stored in memory location 0051, and then to go to memory location 0503 for the next instruction. The third instruction, 02 0052 0504, directs the computer to take the number stored in memory location 0052 to the arithmetic unit and there to add it to the number already in the arithmetic unit and then to go to memory location 0504 for the next instruction. The other coded instructions are interpreted similarly.

The sixten-digit coded instructions are recorded on cards or tape by manual operation of machines designed for that purpose, and are then tested against a manually computed problem. After any possible errors are found and corrected, the program deck, or program tape (if tape is the input medium) is ready to use.

In solving the problem, the program deck or tape is read by the input mechanism, which places the coded instructions in memory registers 0501 through 0506. Data cards containing the values of x and y to be used are placed in the input unit, the memory location of the first instruction to be executed, 0501 in this case, is specified by means of manual controls on the computer, and the computer is started. Having been given the location of the first instruction, the computer automatically executes the whole series of instructions for all pairs of values of x and y and writes out corresponding values of Z. This may be done by means of an "on line" typewriter or printer or by punching cards or tape whichever is the output medium used. If the output is in the form of punched cards or tape, these are used to produce a printed sheet on a line printer or tape actuated typewriter.

While this is a simple case involving only six instructions, it serves to illustrate the programming procedure. A typical engineering problem involves many arithmetic operations and may require a program consisting of several hundred computer instructions covering a complete analysis.

A variety of coding systems is used. In the example given here, a two-address system was used. Each coded instruction contained two addresses, one for the number to be operated on and another for the next instruction to be executed. Single-address, three-address and four-address systems are also used. In the single address system, only the address of the operand is given. Instructions are executed in numerical sequence unless otherwise specified so that it is not necessary to furnish the address of the next instruction. Some two-address systems also use this device, the two addresses being used for the two operands involved in the operation. In the three-address system, two addresses are used for operands and the third to specify where in the memory the result of the operation is to be stored. The four-address system uses two addresses for operands, one for storing the result of the operation and the fourth for the location of the next instruction to be executed.

LOGICAL DECISIONS

The way in which the computer can make logical decisions can be illustrated by its use in computing highway profile grades. The computation of grade elevations at successive stations on tangents is a simple incremental process. On any one tangent, the grade elevation at a station is a constant distance above or below the grade elevation of the previous station. On vertical curves, however, the grade elevation at a station is determined by adding to or subtracting from the tangent grade elevation, the vertical distance by which the vertical curve deviates from the tangent at that point. Thus, in computing profile grade elevations, there are two sequences of operations involved; one for points on tangents and an additional sequence for points on vertical curves. The overall flow chart could be drawn as shown in Fig. 3.

In the detailed program, the test to determine if the station lies on a vertical curve would be made by comparing the station with the stored station of the point of vertical curve (P.V.C.). If it is equal or smaller, the computer has not yet reached the vertical curve and the station is on the back tangent. If it is larger, a second test is necessary to determine if the station is less than or greater than the station of the point of vertical tangent (P.V.T.). If it is equal or less, the station is on the vertical curve and if greater, it is on the forward tangent. This is an abbreviated description of the process but serves to explain how the computer can determine which of two courses of action to follow. By this means, the computer can compute grade elevations for an entire project proceeding along a succession of tangents and vertical curves in a continuous operation.

NUMBER SYSTEM

In this paper, frequent mention has been made of the representation of numbers by code patterns, of the presence and absence of holes or magnetized spots, or of positively or negatively magnetized cores. This method of coding is based on the binary number system in which there are only two symbols, 0 and 1, in contrast to the decimal number system in which there are ten symbols, 0 to 9. In the binary system there are no digits larger than 1, just as in the decimal system there are no digits larger than 9. Because only two kinds of digits are used, binary numbers may be represented by the presence and absence of holes in a card or paper tape, the presence and absence of magnetized spots on magnetic tape or magnetic drums, the presence and absence of electrical pulses in a circuit, the "on" or "off" condition of an electronic switch, the conducting or non-conducting state of an electron tube, and so on.

The binary digit 1 may be represented by one of two such mutually exclusive conditions and the binary digit 0 by the opposite condition. Thus, any number in the binary system can be represented by a pattern of holes and no holes, magnetized spots and non-magnetized spots and other similar devices representing a corresponding combinations of ones and zeros. Similarly, a number can be transferred from one part of the computer to another by transmitting through the electronic circuitry a pattern of electrical pulses and intervals between pulses corresponding to the combination of ones and zeros that represent the number.

In the binary system, a number is represented in terms of successive powers of 2 just as in the decimal system a number is represented in terms of successive powers of 10. For example, the number 302 in the decimal system can be written as:

$$3(10^2) + 0(10^1) + 2(10^0)$$

In the binary system, the same number would be expressed as 100101110 which means,

$$1(2^8) + 0(2^7) + 0(2^6) + 1(2^5) + 0(2^4) + 1(2^3) + 1(2^2) + 1(2^1) + 0(2^0)$$

or in the more familiar decimal equivalents,

$$256 + 0 + 0 + 32 + 0 + 8 + 4 + 2 + 0 = 302$$

From this it can be seen that in either the decimal or binary system, the digits used to form a number actually represent the coefficients of successive powers of the base or radix of the system, with the coefficient of the zero power in the units position. The binary equivalents of the decimal numbers zero to nine are as follows:

Decimal	Binary	
0	0	$0(2^0)$
1	1	$1(2^0)$
2	10	$1(2^1) + 0(2^0)$
3	11	$1(2^1) + 1(2^0)$
4	100	$1(2^2) + 0(2^1) + 0(2^0)$
5	101	$1(2^2) + 0(2^1) + 1(2^0)$
6	110	$1(2^2) + 1(2^1) + 0(2^0)$
7	111	$1(2^2) + 1(2^1) + 1(2^1)$
8	1000	$1(2^3) + 0(2^2) + 0(2^1) + 0(1^0)$
9	1001	$1(2^3) + 0(2^2) + 0(2^1) + 1(2^0)$

This table can be extended to show the binary equivalent of any decimal number. For example, the binary equivalent of the decimal number 3201 is 110010000001 in which the 1 at the extreme left is the coefficient of the eleventh power of 2.

Since the conversion of decimal numbers to binary numbers and vice-versa is awkward, it is handled either through programming subroutines or automatically by electronic circuitry in the computer. It is not necessary for the user of the computer to convert decimal numbers to the binary system; numbers are entered in the computer in decimal form. Also, answers are produced in decimal form. To simplify the internal conversion process, the binary coded decimal system is used in many computers. In this system, each decimal digit is represented individually by a group of binary digits. Various equivalents are used, perhaps the most common of which corresponds to the straight binary system described previously, using four binary digits to represent each decimal digit. Thus, the decimal number 3201, which in the straight binary system was previously shown to be 110010000001, would be 0011001000000001 in the usual binary coded decimal form. The four digits 0001 at the extreme right represent the decimal digit 1, the next four digits proceeding to the left represent the 0, the next four, the 2, and the four at the extreme left, the 3 so that taken together they represent 3201.

In the computer, the arithmetic and control circuits operate on groups of four binary digits, called "bits." Each group represents a decimal digit. The patterns of magnetized spots on tape or drums, of magnetized cores and of electrostatic charges, previously described in the section on components, are arranged in this same way. This method of representation is called the 8-4-2-1 system. Similar systems are in use termed the 2-4-2-1, the "excess-three," the "two out of five" and others. Each system has certain advantageous characteristics and detailed information can be found in the Bibliography. This reference material also provides information on the addition, subtraction, multiplication and division of binary numbers. These processes are similar to those used with decimal numbers but somewhat simpler.

EXTERNALLY PROGRAMMED COMPUTERS

This paper has been directed principally toward electronic computers in which the sequence of operations is controlled by a program of computer instructions stored in the computer's memory. Computers of this kind are called "stored program" or "internally programmed" computers.

The earlier automatic digital computers beginning with the mechanical Analytical Engine conceived by the British mathematician Charles Babbage in 1833 and extending to the ENIAC which was placed in operation in January 1947 all used some external means for controlling the sequence of operations (punched cards, punched paper tape or wired control panels). The memory was used only for storing problem data. These machines are called externally programmed computers.

The EDVAC (Electronic Discrete Variable Automatic Computer), placed in operation in 1949, is believed to be the first stored program computer, at least in the United States. Since then, all of the large and intermediate size computers, as well as most of the small computers, provide for this type of sequence control.

There are, however, a number of small computers that are externally programmed. The most common sequence control device is a wired control panel or plugboard. The wires are connected to metal hubs by plugs or pins that extend through the board and press against contacts in the machine. The ways in which the wires are connected determine the operations to be performed and their sequence. The plugboards are removable and, in normal usage, there will be many of them kept in a wired, ready to use condition, one or more for each type of problem which recurs frequently.

Punched cards are used for entering problem data. A card is read, a group of arithmetic operations is performed on the problem data read from the card, and the answers are punched in the same card. In most cases, as answers are punched in one card, problem data are read from the next card. The computation is completed in the time between reading successive cards. Where many program steps are required in the solution of a problem, two or more passes of cards through the computer are required, each pass using a different plugboard. While minor variations in the sequence of operations are possible through wiring arrangements, this type of computer is much less flexible than the stored program type. The memory is quite small and because of the multiple card passes and the need for changing plugboards, overall operating speeds are relatively slow.

In another type of externally programmed computer, the sequence of operations is controlled by punched cards on which program instructions are recorded. General-purpose control panels wired to perform a number of basic operations are used in conjunction with the cards. As the cards are read, the appropriate arithmetic operations are selected by the punched instructions and executed. Problem data to be operated on is punched into the same cards that contain the instructions. The specified operation is performed as the card is read (instructions are not stored in the memory). Results of the computation may be stored temporarily, punched into cards, or printed in report form by means of an on-line printer. Because the wired control panels are general purpose panels, in order to change from one problem to another, it is necessary to change only the program deck. In an alternative method of operation, special purpose panels, arranged specifically for the problem to be solved are used. This reduces running time for a given problem but requires wiring new panels for each new problem. In either case, since instructions are read from cards and executed as read, the speed of operation is limited to the rate at which cards can be read. Variations in the sequence of operations can be made by selective reading of instruction card columns.

A third type of externally programmed computer is similar to the plugboard computer first described except that sequence control is accomplished by inserting pins in a prewired board. The pattern or arrangement of the pins governs the operations to be performed and their sequence. Data is entered either from a keyboard or by means of punched paper tape. A small magnetic drum-memory is provided for problem data. Results are printed out on a typewriter type of printer.

In still another type of externally programmed computer, program instructions are entered from a keyboard the first time the problem is solved. As this is done, a punched program tape is produced that may then be used as the control for automatically repeating the solution with additional sets of given data. Problem data are entered either manually from the keyboard or by means of punched paper input tape and stored in a small magnetic drum memory. Intermediate results are also stored in the memory. Final results of the computation, and intermediate results if desired, may be typed in report form by an on-line typewriter, punched in paper tape, or both. Wired control panels are provided as supplementary sequence control, primarily for subroutines and logical decisions.

While the externally programmed computers are not as fast or as flexible as the internally programmed computers, their programming is simpler. They can be used for the solution of any problem that can be resolved into basic arithmetic operations and are widely used for both accounting and engineering computations.

APPLICATIONS

As a rough guide, the number of hours required to develop a typical computer program, beginning with the analysis of the problem and extending through the compilation and debugging of the coded program, is about equal to the number of computer instructions involved. The use of the procedures described in the section on automatic coding can shorten this time substantially but even then, the complete programming operations for a problem solution requiring several hundred computer instructions might still extend over a period of several weeks. When the program is completed and placed in use however, the actual computer time for solving such a problem for one set of given values would be only a few minutes, or a few seconds in the large computers.

Because the preparation of the program is so time consuming and costly, it is obviously uneconomical to use the computer for a "one time" type of problem, unless of course, the program could be prepared in less time than it would take to solve the problem manually. The computer is of greatest advantage in the case of frequently recurring problems where program-development costs can be spread over many applications and where, except in the first instance, the program is immediately available for use. Most highway and bridge problems are recurrent and tremendous savings in time and manpower are attainable by using an electronic computer for their solutions.

In earthwork computation, for example, about 80 to 90 man-hr per mile are required using the conventional method of plotting, checking and inking cross sections, spotting grade profile elevations, drawing roadway templates, planimetering areas and computing volumes and mass diagram ordinates. In the electronic computer method, volumes and mass diagram ordinates can be produced at rate of $2\frac{1}{2}$ to 3 hr per mile using an intermediate size computer, including the punching of terrain and template input data on cards or tape. Only approximately 15 min of computer time is required. Also, because the computer results are obtained entirely by arithmetic computation, errors and inaccuracies in plotting, planimetering and transcribing data are completely eliminated. The computer method does not produce drawn cross sections. If they are necessary at critical points, they can be produced rapidly and accurately by an electronic plotter actuated by the computer input cards or tape, or they can be plotted manually.

In other types of problems, even more impressive savings are attained. The computation of the dimensions and elevations needed for laying out, fabricating and constructing multispan curved bridges with variously skewed piers and abutments is tedious and time consuming. In one project of this kind with fourteen spans, a complete solution was obtained on an intermediate size electronic computer in 11 min. Based on similar problems done previously, it was estimated that it would have taken 22 man-days to solve the problem using a desk calculator.

These two examples are typical of the savings in time and manpower that are possible. Even more important however, are the savings in construction costs and highway user costs resulting from the more exhaustive analyses and more refined designs made possible by high-speed automatic computation.

Using an electronic computer, the investigation of several alternate designs can be completed in less time than it would take to make one investigation by other means.

Other computer programs in the highway and bridge engineering fields cover a wide range of applications including highway planning, location and design, soils mechanics, structural analysis and design, traffic analysis, surveying computations, quantity computations, bid analysis, and cost control. Still other programs are in process of formulation.

The heavy burden of program development has been eased substantially through cooperative effort. Users of the same makes of computers have formed user groups that meet periodically to review progress made and plans for further development. Programs developed by one member of a group are made available to other members. Through such program exchange, duplication is minimized and maximum progress is assured within the user group.

Because each computer uses a different machine language or code, program exchange between users of different computers is more difficult. For this purpose, a library of electronic computer programs has been established within the United States Bureau of Public Roads, Department of Commerce (BPR), to serve as a central point for the receipt and distribution of computer programs developed for use in the highway and bridge fields. Programs are contributed by the highway departments, consulting firms, educational institutions, computer manufacturers and others. Each program received is adjusted to remove terminology peculiar to the computer for which it was developed. The final library version of the program is expressed solely in English and mathematical terms in accordance with an established format to assure uniformity of content and complete coverage. It includes a detailed flow chart and is otherwise sufficiently complete and detailed for ready coding for use with any make of digital computer. In addition, library programs are developed within the BPR both directly and through cooperative projects with state highway departments and educational institutions or research organizations. Users are kept informed of the programs available from the library through program listings that are issued periodically.

THE FUTURE

The general tendency in developing computer programs for highway and bridge work has been to follow exactly the procedure used in solving the problem with pencil and paper or a desk calculator. This is one reason that such rapid progress has been made in converting to electronic computation. However, because the computer performs mathematical operations so rapidly, there is much greater latitude in the selection of methods. Assumptions, approximations and rule-of-thumb procedures usually made in analysis and design can be discarded and entirely new approaches based on fundamental principles can be used. In this way, proper consideration can be given to all significant variables and conditions involved in the problem to obtain more refined designs than previously were considered to be economically feasible. There is much room for research and development in this area. In comparison with the potentialities of the electronic computer, applications in the highway engineering field to date (1959), although seemingly extensive, have actually been quite limited.

With respect to the computer itself, the rate of research and development assures continued improvement in operating characteristics. The development of faster memory devices is moving forward rapidly. The trend toward a greater use of transistors is producing substantial reductions in space and power requirements. New materials and devices are being investigated in an effort to develop faster arithmetic units, more flexible input devices and greater reliability.

Even more important to the user, however, is the work that is being done to make the computer easier to use. A great deal of work is being done in the area of automatic coding that was described briefly in this paper. The objective is to develop a system in which the computer will produce a solution from instructions expressed in the natural language of the user, that is, in a combination of words and mathematical terms. In this way, the engineer will be able to use the computer without working through a coder, and of course, the time required to prepare a problem for computer solution will be reduced materially.

CONCLUSIONS

This paper is essentially an introduction to the electronic computer and its use in highway and bridge engineering. No attempt has been made to provide an exhaustive treatise on the electronic computer. Material on flow charting and coding techniques, instruction modification, scaling, error checking, relative programming and other techniques and devices has purposely been omitted Also, no attempt has been made to present electronic circuits or numerical analysis. Information on these and other items that may be of interest to those who wish to pursue the subject further are given in the Bibliography references.

The electronic computer is a powerful tool for the engineer and will become even more powerful in the future. It is important to bear in mind however, that it is only a computing machine. The thinking involved in the solution of an engineering problem must continue to be done by the engineer. Although the computer can call on its memory for information and can make simple logical decisions, it can do these things only when explicitly directed to do them by means of instructions in the program. It will do exactly what it is directed to do; no more and no less.

APPENDIX.—BIBLIOGRAPHY ON ELECTRONIC COMPUTATION

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DISCUSSION

CHARLES E. CARLSON,[2] A. M. ASCE.—There has been presented in this paper a comprehensive survey of the use of an electronic computer in highway and highway bridge engineering. Many engineers are finding computer solutions of tedious problems efficient, but lack a suitable background for understanding how these solutions are developed. The author's article helps to provide this basic knowledge.

To make efficient use of an electronic computer more is needed than an understanding of SOAP II, FORTRAN, or any of the other terms that are new to the civil engineer. The electronic computer is an engineering factory that can produce for the engineer as efficiently and as much as he will allow. For effective use we must understand the dimensions of our future needs. Even at this early stage of computer usage, there are some engineers who are losing the use of some of their expensive programming because the initial outline of their program is not flexible enough. Fig. 4 is a flow chart of pre-program planning that has been developed from the experience of the writer.

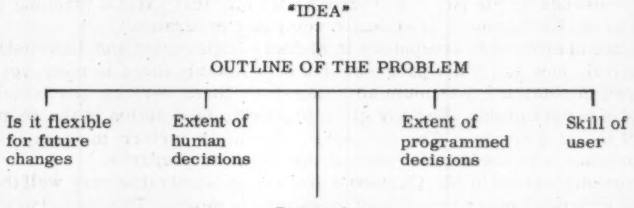
"IDEA"
OUTLINE OF THE PROBLEM
Is it flexible for future changes
Extent of human decisions
Extent of programmed decisions
Skill of user
Economics of use

FIG. 4

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[2] Bridge Design Engr., Wisconsin Highway Comm., Madison, Wis.

As an example of flexible and economical programming, the Wisconsin Highway Commission is developing a chain of programs to aid in the design of either steel composite or concrete slab continuous bridge superstructures. The series makes extensive use of library programs and does not confine an engineer to pre-determined "typed" design. Its dimensions are almost limitless and can be expanded for greater use or used in "pieces." The programs are shown in Fig. 5.

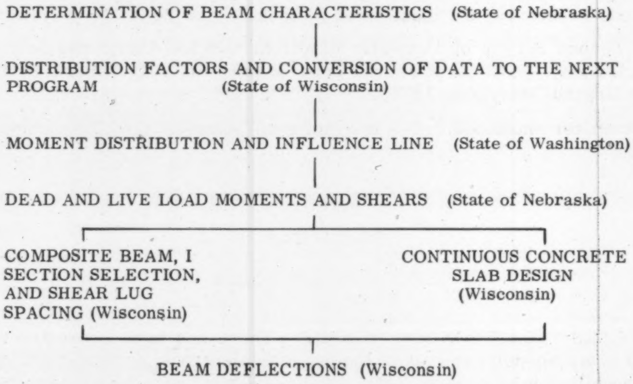
DETERMINATION OF BEAM CHARACTERISTICS (State of Nebraska)
DISTRIBUTION FACTORS AND CONVERSION OF DATA TO THE NEXT PROGRAM (State of Wisconsin)
MOMENT DISTRIBUTION AND INFLUENCE LINE (State of Washington)
DEAD AND LIVE LOAD MOMENTS AND SHEARS (State of Nebraska)
COMPOSITE BEAM, I SECTION SELECTION, AND SHEAR LUG SPACING (Wisconsin)
CONTINUOUS CONCRETE SLAB DESIGN (Wisconsin)
BEAM DEFLECTIONS (Wisconsin)

FIG. 5

To re-emphasize the need for prior planning, the writer has outlined the four basic steps for successful engineering programming:

1. Ascertain the extent of the problem. Find out what is wanted and future needs;
2. Investigate library sources to avoid work duplication;
3. Prepare cost and time estimate; and
4. Establish methods and techniques.

L. R. SCHUREMAN,[3] **M. ASCE.**—The writer appreciates Mr. Carlson's interest in discussing his paper and agrees with him that careful planning is essential to the development of effective computer programs.

The use of electronic computers in highway engineering and administration is relatively new and while progress has been notable there is much yet to be developed in obtaining maximum advantage from these devices, particularly in the field of programming. For any given problem, the solution is the responsibility of the programmer. The computer will simply perform the computations in accordance with the detailed plan of operation he prepares.

The example given in Mr. Carlson's discussion illustrates very well the way in which programs may be combined to operate in series. This has also proved effective in highway design where several component programs are combined and arranged so that the output from one serves as input for another with provision for review and decision by the engineers as necessary during the overall process.

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[3] Highway Engr., U. S. Bur. of Public Roads, Washington, D. C.

With the larger internal storage capacities becoming available in computers, correspondingly greater opportunity is afforded for increased flexibility and for the complete solution of an engineering problem in a single computer run, as opposed to a series of programs used in sequence. It is essential however, regardless of the type of problem involved, to plan and develop the program so that the computer adequately serves the engineer, providing him with the information he needs for sound engineering decisions.

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TRANSACTIONS

Paper No. 3185

COUNTY MOTOR VEHICLE TRANSPORTATION SYSTEM PLAN

By Victor W. Sauer,[1] F. ASCE

SYNOPSIS

Contra Costa County, Calif. has developed a plan for a road network serving present and future needs. As a technical feature, traffic data are coded on a grid system basis, thus facilitating re-use of information. The cooperation of the various organizations involved and the attempts to obtain public support are emphasized. Contracts stating fields of investigation and State Senate resolutions are cited.

INTRODUCTION

A plan for a county motor vehicle transportation system should be developed without regard to political boundary lines. It should be financed on the basis of public benefit, and the planned system should be built on priority of needs. These are the steps necessary to achieve a balanced and integrated system of state highways, county roads, and city streets to best serve the highway user and to encourage optimum social and economic development of the county.

A plan for a county motor vehicle transportation system can be considered as a dream set forth in a report or projected on the drawing broad. A system is a dream come true. From dream to plan to system requires cooperation, coordination and communication among all groups who must accept the plan and approve its transformation into a system by the appropriation of necessary funds.

In California there are many road plans completed (1959) through the combined efforts of the state, the counties, and the cities. Many persons are now aware that past frictions among different groups interested and concerned

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Note.—Published essentially as printed here, in September, 1959, in the Journal of the Highway Division, as Proceedings Paper 2144. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Public Wks. Dir. Contra Costa County, Martinez, Calif.

in different motor vehicle transportation systems stemmed, many times, from prides and prejudices, but more frequently from lack of funds. There has been a tremendous change toward real cooperative effort. This somewhat recent metamorphosis is due in part to the recognition that the democratic concept of "the greatest good for the greatest number" has a logical application to road matters. Applying this concept to transportation systems, Table 1 ranks several systems within Contra Costa County, Calif. in the order of their importance. However, these systems are inter-related and it is essential that the planning for them be done cooperatively in order for each to attain maximum utility.

TABLE 1

Priority	System
1	Interstate System
2	California Freeway System
3	Contra Costa County Freeway-Expressway-Arterial System
4	Contra Costa County Feeder Road System
5	Contra Costa County Residential Road System

The foregoing comments imply the necessity for having a solid basis for road planning. In Contra Costa County this basis rests on four major premises:

A. Traffic is no respector of political boundary lines.

B. There should be general financing for general needs and local financing for local needs.

C. Construction should proceed on priorities based on need.

D. Motor vehicle transportation systems yield important economic benefits other than time and distance savings.

GEOGRAPHY AND HISTORY AFFECTING THE PLAN

Before presenting the Contra Costa County plan, it is desirable to describe Contra Costa County itself and to review the events that preceded the development of the plan. This will permit the plan to be viewed in its physical and historical setting for a better total understanding of its meaning.

Geography.—The county is bounded by Alameda County, San Francisco Bay and San Pablo Bay on the west; by Suisun Bay, the Carquinez Straits, and the San Joaquin River on the north; by San Joaquin County delta land on the east; and again by Alameda County on the south. The western half of the county is hilly, being dominated by Mt. Diablo. The western ranges, composed of the Berkeley Hills, the San Pablo Ridge, and the Pinole Ridge, run north to south separating the city of Richmond and the Alameda County area from the central and eastern Contra Costa County cities. These hills pose problems since they are a barrier to traffic flow between these densely populated areas. The basin between the Berkeley Hills and Mt. Diablo comprises the Walnut Creek watershed, with certain severe drainage and slide problems occurring in the winter months. Victor A. Endersby, F. ASCE, has said,[2]

> "The County is remarkable for extreme soil variations with sudden changes and a great many different formations. Water may be struck almost anywhere during heavy construction; and a large number of springs and water seams are bone dry during the late summer so that in winter a spring may show up where it would not be suspected during dry season construction. Meterologically, the county's climate is a front between the Pacific Ocean climate and the Sacramento-San Joaquin Valley climate."

History.—Historically, the first of the county's industrial and trade centers were located along the water route on the north, forming the major cities along the shore line (Richmond, Martinez, Pittsburg, and Antioch). The first railroads paralleled this route as well as the first highways. The interior of the county was rural, with grain its chief commodity. Transportation from farm to port was by wagon over roads radiating from the interior to the coast line. Little access was provided directly from the San Francisco-Oakland Bay area over the hills to the central portion of the county. This resulted in a slow growth of the presently heavily populated area of Lafayette-Walnut Creek-Concord-Danville, in the central part of the county. This area's potential for homesites and for recreation was recognized early, however, and was opened for these uses by the construction of the Caldecott (Broadway low-level) tunnel through the hills to Oakland and Berkeley, in 1938. The overflow of population from the heavily populated Bay area literally flooded through this portal after World War II. The area is still growing with new residential subdivisions mushrooming throughout. The new population consists heavily of commuters whose traffic chokes the highways during peak hours and, to support the increased population, trucking has been steadily increasing.

Fortunately, the impact of these changes affecting roads fell on a local government already partially equipped to meet it. To illustrate this and to provide the historical setting for the Contra Costa County Motor Vehicle Transportation plan, several of the background events leading to the plan must be introduced.

The First Contra Costa County Road Bond Issue.—In July, 1919, the county passed a $2,600,000 road bond issue for the improvement of 100 miles of county roads. Today (1959) we can still ride over a considerable mileage of these roads. Much of the remainder has disappeared by incorporation into subsequent improvements. This 1919 bond issue stimulated an interest in roads that remains today.

The Collier-Burns Act of 1947.—The wisdom of the California State Legislature in passing the Collier-Burns Act of 1947 becomes increasingly obvious with the passage of time. An important feature of this Act provided for proper county road administration: "Policies will be determined by the Board of Supervisors and carried out by the Road Commissioner." The Act also benefited the California Division of Highways. The consequent achievements of the Division have rightly drawn state and national attention, and because of the relative magnitude of the Division's activity, the tremendous importance of the Collier-Burns Act to California counties in road administration and financing is sometimes obscured.

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[2] Presented at the 1954 annual meeting of the Inst. of Transportation and Traffic Engineering, Univ. of California, Berkeley.

The Contra Costa County Road Bond Issue of 1952.—By the early 1950's the cooperation, coordination, and communication among the various groups interested in better roads again resulted in a county road bond issue, this time for $10,250,000. This 1952 bond issue was predicated on the removal of the then urgent, non-deferrable deficiencies measured in terms of structural, capacity, and safety ratings. Approximately 78 miles of county roads, a railroad underpass, and some bridges were approved for construction. About $8\frac{1}{2}$ miles of new roads were included where the traffic benefit and service potential were obvious, but in general there was no plan for future traffic-ways with detailed right-of-way requirements.

The taxpayers were promised better roads and reduced taxes if the bond issue was approved and these promises have been kept. The bond issue is being paid off with cheaper money than was borrowed, a progressively broadened tax base is being developed, and there is the immediate benefit of huge savings for the highway user. For example, the time and distance savings for the Cummings Skyway were $909,000 in 1 yr while the total cost of this bond issue project was only $1,434,210.

The taxpayers also have found it a matter of interest and satisfaction that since 1953, for normal years without floods, there has been a large reduction in cost per mile for road maintenance resulting in an average saving of more than $400,000 a year. A part of the savings is due to a reduction in the road maintenance force by sixty-seven men. Another savings resulted from the disposal of equipment having a book value of $165,000. These savings were made despite the fact that there are 104 miles more of road to maintain in 1958 than there were in 1953.

This 1952 bond issue clearly demonstrated the direct and immediate cash benefit of good cooperation, coordination and communication among the technical staffs, taxpayers, and official groups in our county. The term "technical staffs" used herein and elsewhere includes engineers, planners, and traffic specialists, all of whom must work together as a team.

The Local Road and Street Deficiency Reports.—Local road and street deficiency reports have been required every 2 yr since 1952 for the California Joint Interim Committee on Transportation Problems. The preparation of these reports provided sufficient stimulation to pull together more strongly than before city and county technical staffs. An extra dividend of this mutual effort was the creation of the City-County Engineering Advisory Committee. The year 1952 showed a deficiency of $37,000,000, 1954 showed $35,000,000, 1956 a deficiency of $34,000,000, and 1958 of $37,000,000. The standards used for all reports have been essentially the same. The increase from 1956 to 1958 is primarily due to increases in traffic and to consideration of all deficiencies in 1958 where previously only "critical" deficiencies were listed.

The United States Bureau of Public Roads "210" Report.—Section 210 of the Federal Highway Act of 1956 required each state to provide an estimate of the cost to remove the existing and future deficiencies on state highways, county roads, and city streets for the 15-yr period from 1956 to 1971. Every city and county in the state voluntarily made and submitted its own report to the California Division of Highways. The work on the "210" report improved participation at policy making, technical, and administrative levels and strengthened the concept that "traffic is no respector of political boundary lines."

The Senate Concurrent Resolution No. 26 of the 1957 State Legislature.—This resolution required a study that resulted in the proposed California Freeway System and provided, as well, a sound basis and a prerequisite for all future cooperative motor vehicle transportation planning. For these reasons it was imperative that the California Freeway System be adopted and built on schedule (Appendix).

Contract provisions of Senate Concurrent Resolution No. 26 permitted and helped finance local city-county-state cooperative thoroughfare or traffic-way studies. Either a city or county could be the contracting agency. Contracts with both the consultant and the state were necessary (Appendix). From the city-county-state cooperative road studies the state received the data needed and required by the resolution, and the cities and counties obtained the shape and location of tomorrow's local traffic-ways.

The Joint Interim Committee on Highway Problems Created by Senate Concurrent Resolution No. 134 of the 1957 State Legislature.—This legislative committee held statewide hearings on the proposed California Freeway System. It is of great significance that eleven times Senate Current Resolution No. 134 specifically indicates that city streets and county roads are of interest and concern to the State Legislature. The Legislature has increased again the interest in local motor vehicle transportation planning (Appendix).

The County Setback Ordinance.—In the application of this ordinance to protect the rights of way for roads of tomorrow, it has been found desirable to explain the proposals. Conferences with representatives of cities, industries, Chambers of Commerce, and civic leaders in different areas have resulted in agreement. The only unanswerable question is: "How soon can you improve this road?" This setback program is, in effect, creating area demands for better roads. The sum of all area demands would be equal to an entire future county demand.

The Rotating Right-of-Way Fund.—A newspaper reporter suggested that the county develop a rotating right-of-way fund similar to the California Division of Highways' fund that has been so successful. The idea was given wide publicity and received widespread favorable comment and support.

Contra Costa County sponsored legislation, passed by the State Legislature, permitting counties to establish such funds. The prospects are good for the near future creation of this fund, that has a limitation of $\frac{1}{2}$% of the county's assessed valuation.

The Mayors' Conference.—The Mayors' Conference, formed in June, 1951, is composed of the mayors of the county's eleven cities. It has been instrumental in developing unity among city governments and in improving city-county relationships. If, and only if, there is cooperation among the policy and law-making bodies of the county, can cooperation among the technical staffs be, in the first place, possible, and in the second place, effective.

The City-County Engineering Advisory Committee of Contra Costa County.—Initiated by the Mayors' Conference, this committee was endorsed by the Mayors' Conference, the Board of Supervisors, and by each individual city. The broad purpose of the committee is "to promote mutual interest and coordination of effort in city and county public works matters and to make appropriate recommendations to our respective City Councils and to our Board of Supervisors." The official recognition given the cities' and the county's engineering staffs hastened the development of the philosophy of a City-County Thoroughfare System.

The Adoption of the Philosophy of a City-County Thoroughfare System by the Cities and the County.—The Mayors' Conference and the Board of Supervisors directed the City-County Engineering Advisory Committee to develop a plan for the improvement of city streets and county highways. The idea for a City-County Thoroughfare System resulted and was approved by all the policy-making bodies concerned.

It is to be emphasized that this idea for a City-County Thoroughfare System was a fundamental concept and was not an engineering plan for precise or future projected routes, or finalized rights of way and geometric requirements.

The City-County-State Cooperative Road Study.—Under the provisions of Senate Concurrent Resolution No. 26, the county, with the approval and support of the cities, entered into contracts for a road study that is now complete and is being carefully reviewed.

Conducted by an engineering consultant, this study has made a reality of the City-County Thoroughfare concept adopted by the Mayors' Conference, and the Board of Supervisors, and the individual cities. The City-County Thoroughfare System will be a selected portion of the system of freeways, expressways, and arterials developed in the cooperative Senate Concurrent Resolution No. 26 study.

The foregoing background of events demonstrates the growth toward a high measure of solidarity and unity of purpose in road matters in Contra Costa County.

PLAN FOR A MOTOR VEHICLE TRANSPORTATION SYSTEM

Purpose and Scope.—The county's plan for a network of traffic-ways to serve the entire future traffic needs of the county has a primary target year of 1980, for design purposes. Supplementary design data were developed for the secondary target year, 1963, to show which elements of the 1980 system had the highest priority and should be constructed first.

However, the rights-of-way requirements were developed for ultimate population conditions of the county. "Ultimate" is defined as the time at which the optimum development of all individual areas of Contra Costa County occurs. The years 1963 and 1980 represent estimated percentages of population of the "ultimate year." The "ultimate year" can arrive in 1970, or 2000, or some other year, depending on the rate of population growth. However, with ultimate rights-of-way requirements obtained or protected, there is no problem.

The core of the plan is this: It gives the shape and location of tomorrow's roads, the freeways, expressways, arterials, and important feeders that will comprise Contra Costa County's future motor vehicle transportation system. The recommendations for new or expanded existing facilities are presented on general maps and strip maps, with major urban areas depicted on individual maps. Fig. 1 shows the freeways, expressways and arterials for the target year 1980. Routes for proposed freeways and expressways had sufficient detailed study to insure that the facilities could be built to required standards along the general, but not precise, alinement indicated on the plan. The importance of the feeder system of the plan was recognized by the consultant who made a distinction between feeders of general county interest and those of local area interest.

Time Element and Costs.—The study took 19 months to complete at a cost of $160,000, exclusive of the cost for right of way estimating and detailed topo-

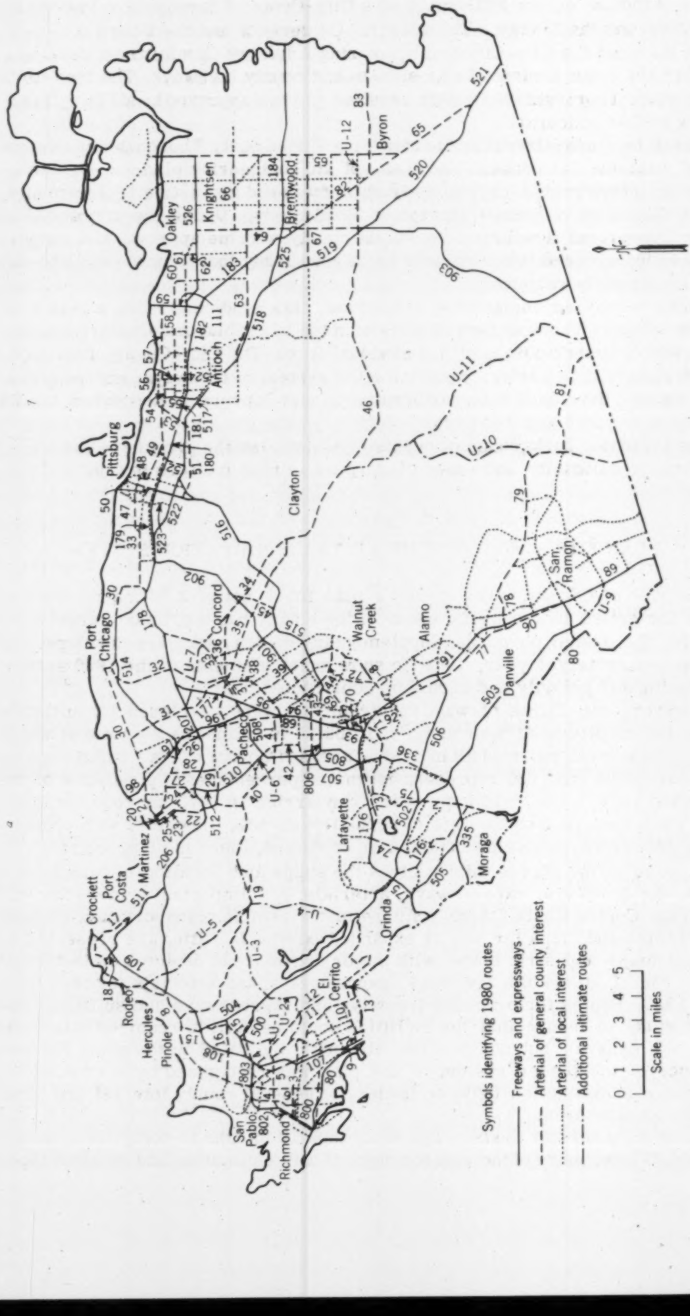
Richmond
San Pablo
Pinole
Hercules
Rodeo
Crockett
Port Costa
Martinez
Port Chicago
Pittsburg
Antioch
Oakley
Knightsen
Brentwood
Byron
Concord
Pacheco
Clayton
Walnut Creek
El Cerrito
Orinda
Lafayette
Moraga
Alamo
Danville
San Ramon
Symbols identifying 1980 routes
Freeways and expressways
Arterial of general county interest
Arterial of local interest
Additional ultimate routes
0 1 2 3 4 5
Scale in miles

FIG. 1.—SYSTEM OF FREEWAYS, EXPRESSWAYS, AND MAJOR STREETS RECOMMENDED FOR CONTRA COSTA COUNTY

graphy, both of which were supplied by Contra Costa County. The cost was equally borne by the cities, the county, and the state.

Administration and Non-Technical Highlights.— The county agreed to provide all necessary aerial photographs, survey information, and estimates of rights-of-way costs. Because the necessary amount of photography and surveying was an intangible, it was difficult to include such items in an equitable contract. The Right of Way Division of the Public Works Department was in an excellent position to supply the necessary right-of-way information by virtue of its thorough and intimate knowledge of county land values.

The Public Works Department was named in the contract as coordinator for the study. The Public Works Director appointed a coordinating committee consisting of his deputy, as chairman, the traffic engineer and a member of the Advance Planning Section of the Planning Department. This committee held many regular and special meetings with the consultant's staff and reviewed his monthly written progress reports, which were required by the contract.

Meetings of the City-County Engineering Advisory Committee were devoted to studies and explanations of different phases by the staff of the consultant.

The Public Works Department organized, and was the headquarters for the campaign to keep the general public informed of the study and to secure needed data from highway users. The highway users cooperated well and contributed vitally to the report by completing and returning more than 37,000 of the 146,000 postal card questionnaires mailed on the internal origin, destination survey part of the study. The press, radio, television, and billboard industries, plus labor organizations, the Chambers of Commerce, the Development Association, the Industrial Association, and other groups joined in informing the public of the desirability and necessity for returning the questionnaires. Feature stories, editorials, spot announcements, window posters, leaflets, and huge billboards were used to bring the message to the public.

Methods and Conduct.— All pertinent available data were collected and analyzed. These data included road inventories, origin and destination studies, twenty-six previous significant reports, traffic volume counts, population records and projects, and many other related studies and sources of information.

A great deal of information on traffic volume data was made available by the state, the county, and the cities. Over a thousand individual traffic counts made in 1956, 1957, and 1958, were reviewed, and counts going back as far as 1946 were checked to determine growth trends.

The collection of new data involved four phases of study to bring existing information up-to-date and to provide the required additional data to permit detailed analysis of Contra Costa's traffic-way needs for the present and the future.

Traffic Volume Counts.

Origin and Destination Surveys.—Since labor costs generally have risen sharply since the end of World War II, the most desirable type of origin and destination study, namely the home interview, has become unfeasible. For this reason, the study turned to postal cards for the internal survey.

The Highway Planning Survey Department of the California Division of Highways some time ago adopted a grid system for the entire state. In addition to coding the information from the internal survey by zones, it was decided, on the recommendation of the consultant, to assign to each trip-terminal a pair of coordinates relating it to the planning survey grid. Although the planning survey usually is based on 1 mile or 0.1 mile intervals, for the purposes of this study a 0.2 mile grid was found desirable and was used.

Although this is a much more difficult, time-consuming, and expensive way of coding trips, a number of advantages were gained. For one, information on the trip terminals is now available within a 0.2 sq mile area and in any future rerun of the data, the study would not necessarily be tied to zone boundaries. This is especially important in areas of rapid growth where the present zones may not correspond with the need of future studies because, with this device, the zone boundaries can be changed to fit the conditions. Secondly, through the use of this grid, studies in adjacent counties can easily be correlated. Eventually the entire state could be tied together.

The usual "desire line" method of showing the results of an internal origin and destination study results in such a mass of lines that, unless they are severely culled for the low-intensity desires, this sheet becomes almost unintelligible. For this reason, it was decided to show the results as traffic contours, a technique developed by the California Division of Highways. The contours indicate the intensity of trip desires at any one point. This method has one drawback in that it shows only the traffic concentration at any one point, but not where this traffic comes from or wishes to go. To overcome this difficulty, supplementary contours on a 1 mile grid were prepared for the entire county on a directional basis. This made it possible to identify desires in, for instance, northwesterly or northeasterly directions. This method then shows not only traffic concentration but also direction.

Population Potential and Projected Land Use.—For any given area of the county a change in population or land use will change the need for facilities and provisions for the movement of goods and people. In projecting highway needs it is essential that there be an integrity in zoning and that projections can be made of predictable factors that affect vehicular travel requirements. The most important of these factors are as follows:

1. the number of people who can reasonably be expected to live in Contra Costa County under the optimum condition of development for a balanced land use plan;
2. the stage of development that can be realized by 1980 and 1963, the primary and secondary target years. (Table 2 shows the projected population in number and per cent of growth for key years);

TABLE 2

Year	Population	Optimum Growth
Ultimate	1,297,000	100
1980	842,000	65
1963	477,000	37
1958	375,000	29

3. the distribution of population by areas within the county; and
4. the location of commercial and industrial centers of traffic attraction for the target years 1963 and 1980.

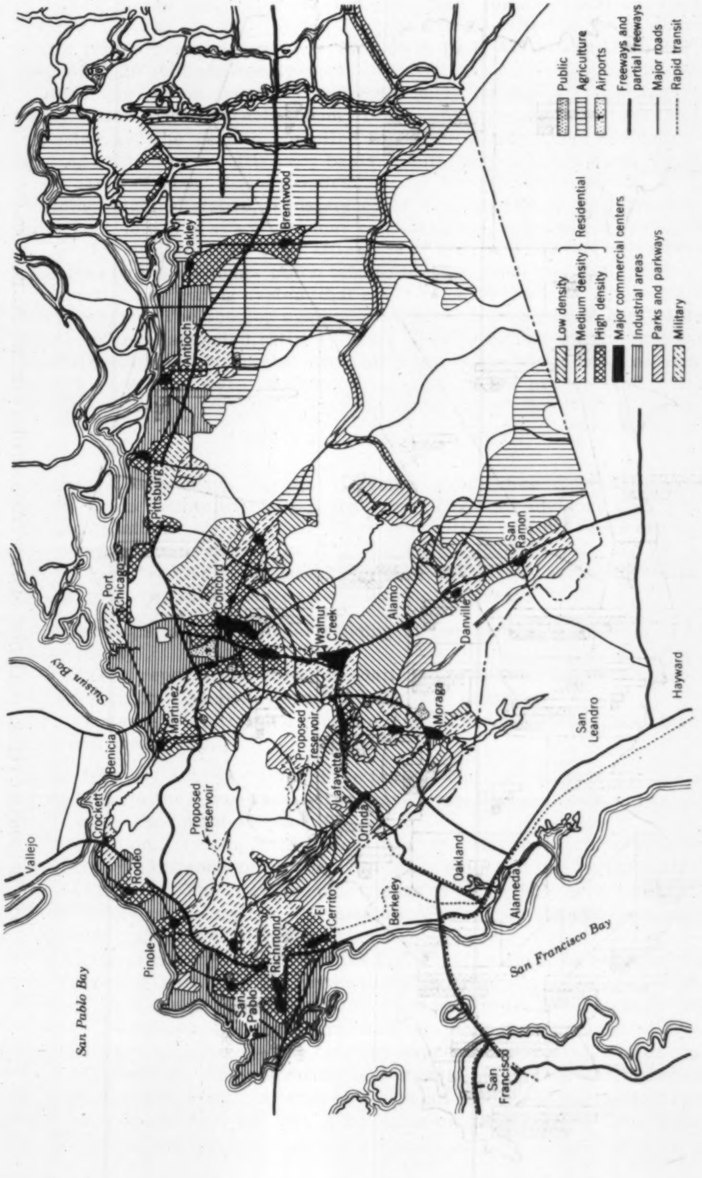
San Pablo Bay
Vallejo
Crockett
Benicia
Suisun Bay
Martinez
Port Chicago
Pittsburg
Antioch
Oakley
Brentwood
Concord
Walnut Creek
Alamo
Danville
San Ramon
Moraga
Proposed reservoir
Lafayette
Orinda
Proposed reservoir
Rodeo
Pinole
Richmond
San Pablo
El Cerrito
Berkeley
Oakland
Alameda
San Francisco Bay
San Francisco
San Leandro
Hayward
Low density
Medium density
High density
Residential
Major commercial centers
Industrial areas
Parks and parkways
Military
Public
Agriculture
Airports
Freeways and partial freeways
Major roads
Rapid transit

FIG. 2.—PROJECTED LAND USE

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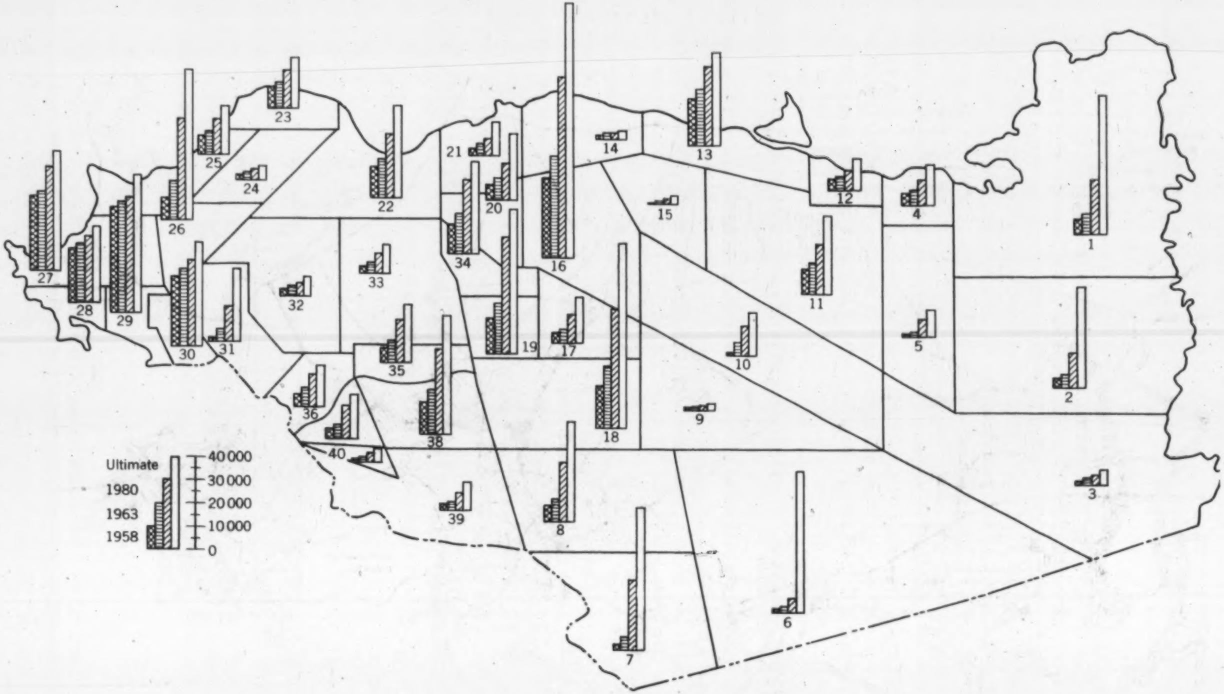
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27
28
29
30
31
32
33
34
35
36
38
39
40
Ultimate
1980
1963
1958
40000
30000
20000
10000
0

FIG. 3.—PROJECTED POPULATION DISTRIBUTION WITHIN CONTRA COSTA COUNTY

Fig. 2 shows the projected land use (1980) for a projected population of 774,000. In Fig. 3 the O and D zones are identified by the numbers. The county population estimates are 375,000 in 1958, 477,000 in 1963, 842,000 in 1980 and 1,297,000 ultimately.

Road and Street Inventory and Sufficiency Ratings.—Three hundred and eighty miles of city streets, county roads, and state highways were carefully inspected and the results compiled to permit a subsequent evaluation of the existing system on a sufficiency basis. "Sufficiency" in this sense consists of three parts of about equal importance; (a) structural conditions, (b) safety, including sight distance, consistency of construction to uniform standards throughout length of roadway under consideration, and adequacy of width, and (c) service, which reflects the efficiency of the route in serving traffic and is measured by the elements which tend to inconvenience the motorist.

A sufficiency rating system that could be of continuing value to the county was developed. The scoring system was derived from the most applicable portions of several accepted rating systems. Specific ratings were established for local streets, major streets, two-lane rural roads and multi-lane highways. Numerical values were assigned these ratings for present and future use.

THE FUTURE OF THE PLAN

Contra Costa County Highway Advisory Committee Recommendations.—Contra Costa County will look to its County Highway Advisory Committee for recommendations on the plan. This committee of fifty members includes broad representation from local government (eleven mayors and two supervisors), and leaders from labor, business, industrial, agricultural and civic groups. The committee serves the following purposes:

1. Develops a better understanding of our highway problems within the entire county.
2. Makes reports and recommendations to the State Highway Commission or Joint Interim Committees on State Highway matters.
3. Works as a unit toward common city and county goals rather than from individual viewpoints.

The Highway Advisory Committee is reviewing the plan to foster agreement on the details of a freeway-expressway-arterial system, which includes the City-County Thoroughfare plan.

After committee approval, each city, the Mayors' Conference, and the Board of Supervisors must adopt the plan before it can be made effective. (The City-County Thoroughfare plan has, as of May, 1961, been adopted by the eleven cities of the county, the mayors' conference, the Highway Advisory Committee and the Board of Supervisors).

A listing of construction priorities for action by 1963, as recommended by the consultant will also be reviewed by the Highway Advisory Committee, and the Committee's recommendations also will be presented to each city, the Mayors' Conference, and the Board of Supervisors for adoption.

Financing Suggestions.—The freeway-expressway-arterial system, including contingencies, administration and rights of way is estimated to cost $250,000,000. The additional cost of the county feeder and residential systems raises the total to $509,000,000. A financing program must be determined and

additional revenues obtained since present funds are insufficient to handle Contra Costa County's existing and future road needs. Table 3 presents a suggestion on equities in financing, and Table 4 gives an overall view of the road deficiency problem in Contra Costa County and makes several financial suggestions for additional revenues to remove the deficiencies.

***Additional Economic Values of Roads*.— It is obvious that substantial increases in funds are needed to liquidate the deficiencies on Costra Costa County's roads and to place the plan in effect. For roads to get a fair and necessary share of available monies it is essential that what may be new perspectives on the value of roads be emphasized. In addition to time and distance savings there are other benefits obtained from roads. Two of these benefits are the broad general economic and the serviceway concept.**

TABLE 3.—COUNTY FUNDS–SOURCES AND POSSIBLE DISTRIBUTION

Existing Source	Distribution		
	General Need;		Local Need;
	Freeway Expressway Arterial System	County Feeder Road System	County Residential Road System
FROM STATE			
Highway Users Revenues	X	X	
"In Lieu" Funds	X	X	X
FROM FEDERAL GOVERNMENT			
Federal Aid Secondary Funds (Including State Matching Funds)	X	X	
LOCAL SOURCES			
Property Burden County-wide			
General County Property Tax	X	X	
Bond Issue (Gen'l Prop. Tax to retire)	X	X	
Unincorporated Areas			
Assessment Districts	X	X	X
Supervisor Road Dists.			X
Sales Price New Homes		X	X
Permanent Road Divisions			X
Out - of - Pocket			
Sales Tax[a]	X	X	X
Fines	X	X	X
Private Financing	X	X	X

[a] Sales tax monies now deposited in County General Fund. If sales tax revenue should be used for road purposes, then property tax rates in Preliminary Financial Study would be reduced correspondingly.

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TABLE 4.—PRELIMINARY FINANCIAL STUDY

$509,000,000*	DEFICIENCIES	REVENUE SOURCES	REVENUE AMOUNTS	NOTES:
COUNTY RESIDENTIAL ROAD SYSTEM	Maintenance 26 Million	17¢ Prop.Tax Unincorp.Area	26 Million	1. Study covers period 1958 to 1980. 2. Tax figures based on projected averages. 3. Study used present City Limits. 4. Study used present prices. 5. Cost estimates for construction include right of way, contingencies and administration.
	Construction 123 Million	Property Burden Unincorporated Areas Assess. Dists. or Other	67 Million	
$360,000,000*		Subdivision Development - Sale Price of Homes	56 Million	
COUNTY FEEDER ROAD SYSTEM	Maintenance 14 Million	Subdivision Development - Sale Price of Homes	43 Million	County General Tax and/or Gasoline Tax to raise 93 Million
$273,000,000*	Construction 73 Million	No Existing Revenue - 44 Million Deficit	93 Million Deficit	
CITY AND COUNTY FREEWAY, EXPRESSWAY AND ARTERIAL SYSTEM	Maintenance 23 Million	No Existing Revenue - 49 Million Deficit		
	Construction 250 Million	Federal Aid	12 Million	
		Traffic Fines	12 Million	
		Highway Users Allocations to County	118 Million	
		In State's Proposed Freeway System ---- Present City-County Jurisdiction	82 Million	

SELF HELP

PLAN A

GEN'L. COUNTY TAX OF 37¢ -$93,000,000

PLAN B

GEN'L. COUNTY TAX OF 27¢ -$68,000,000
BOND ISSUE PROP. TAX 12¢ -$25,000,000
$93,000,000

SELF HELP & STATE AID

PLAN C

GAS TAX 5/8¢ -$54,000,000
GEN'L. COUNTY TAX OF 15¢ -$39,000,000
$93,000,000

PLAN D

GAS TAX 5/8¢ -$54,000,000
BOND ISSUE PROP. TAX 12¢ -$25,000,000
GEN'L. COUNTY TAX OF 5¢ -$14,000,000
$93,000,000

STATE AID ONLY

PLAN E

GAS TAX 1.1¢ -$93,000,000

* Cumulative Deficiency Total

The Broad Economic Benefit.—Robinson Newcomb has said,[3] "Adequate highways increase the gross national product by making it possible to move a greater volume of goods economically.*** The economy cannot grow without adequate traffic facilities."

The Serviceway Concept.—County roads as well as city streets are serviceways provided for the public in general. These serviceways provide the means of transportation of vehicles, pedestrians, and utility services, such as electricity, telephone, water, gas, sewers and refuse disposal. Roads, then, handle more traffic than just goods and people, who legally can be on foot, horseback, or even piggy-back. The same citizen who prefers motor vehicles to the latter three forms of transportation also wants better services in life, such as electric lights, dishwashers, garbage disposal units, telephones, hot and cold running water, inside toilets connected to sewers, and garbage service.

TABLE 5

	Fiscal Year 1958-1959		
	Cost	Tax Rate	Cost Per Month
All School District Taxes	$199.92	$4.998	$16.66
Special District Taxes	86.52	2.163	7.21
County General Taxes	91.60	2.290	7.63
Total Taxes	$378.04	$9.451	$31.50
Average Total Bill for Services (Utilities)	$351.00	$8.775[a]	$29.25
Home Owners Gross Road Budget[b]	$ 33.57	$0.839[a]	$ 2.80
Less Share of Fines, Gas Tax, In Lieu	-18.72	0.468[a]	-1.56
Home Owners Net Road Budget	$ 14.85[c]	$0.371[a]	$ 1.24

[a] Tax rate equivalent
[b] Contra Costa County's 1958-59 Road Budget is $6,222,047
[c] From County General Fund

These low cost, high quality services are excellent, and where they properly use the road rights of way, they should continue to be permitted to do so. There is no implication that the home owners' county road budget should equal the home owners' service budget. However, it seems fair to emphasize that in terms of relative needs of the jobs that have been done, and remain to be done, in terms of services and roads, the road program has not had an equitable and necessary share of funds.

This thought is illustrated by Table 5, showing the relative costs of taxes, utility services and roads for the owner of an average central Contra Costa County home, with an assessed valuation of $4,000.

CONCLUSIONS

Today, Contra Costa County has an excellent report setting out a plan for an integrated and balanced network of state highways, county roads, and city streets. To communicate this plan countywide may take several months. This communication will be successful because the past communication efforts have set the stage for general acceptance of road programs reasonably in line with the desires of the public.

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[3] "Highway Program is Big—But It's Not Big Enough," by Robinson Newcomb, Engineering News-Record, June 19, 1958, pp. 38-43.

However, the mere acceptance of the plan does not necessarily mean immediate action. The huge cost of the plan must be met. Several alternate means of financing have been suggested and there are other means that must be considered.

The county's citizens, who have traditionally supported a strong road program, will continue to provide essential backing for the development of an improved motor vehicle transportation system. This strong support will not be weakened by the knowledge that they can get their ultimate road needs met at no greater additional cost than they now pay to have their garbage removed.

APPENDIX

AGREEMENT FOR CITY - COUNTY - STATE COOPERATIVE ROAD STUDY BETWEEN CONTRA COSTA COUNTY AND TUDOR ENGINEERING COMPANY

THIS AGREEMENT, made and entered into this 23rd day of July, 1957, by and between the COUNTY OF CONTRA COSTA, hereinafter referred to as the County, and TUDOR ENGINEERING COMPANY, hereinafter referred to as the Engineer.

WHEREAS, the County is desirous of having a study made which will be an over-all countywide plan of freeways and expressways, such study to include State, County and City elements, and to locate the potential freeway and expressway routes of such a Countywide system and the necessary connections thereto as nearly as is practicable in advance of detailed engineering design of the projects.

NOW, THEREFORE, it is hereby agreed as follows, to wit:

1. The County hereby employs the Engineer and the Engineer agrees to and accepts such employment to perform the following services upon the following terms and conditions and in consideration of the following payments.
2. The Engineer shall proceed without delay to perform the following work:
   A. Collect all available data pertinent to the study, including:
      1. Inventory of State and County highways and principal city streets that are feeders to the highway system,
      2. Traffic volume counts made within and adjacent to the County boundary,
      3. Traffic origin and destination counts
      4. Accident records,
      5. Population of the County and communities, past and present
      6. Automobile registration of the County, past and present
      7. Industrial growth of the County,
      8. Planning studies and zoning for land use within the County,

9. Planning studies for future highway and principal street developments within the County,
10. Current construction cost records in the area,
11. State and County highway design standards.

B. Collect essential new information and data including:
   1. Limited additional volume counts of traffic, as needed,
   2. Make an inventory of highways, roads and streets, as needed, including both physical characteristics and traffic characteristics. Provide, through speed and delay studies and capacity calculations, a measure of the ability of the existing roadway network to handle traffic loads and show areas of excesses and deficiencies,
   3. Make a comprehensive over-all origin and destination study showing present and future traffic patterns of traffic between internal zones, internal zones to external areas, external areas to internal zones and through traffic. The origin and destination study to be checked by a screen line so that accuracy will meet standards set by the Bureau of Public Roads,
   4. Make land use studies and plans for areas of the County not already so covered. Review existing plans for possible changes in land uses. Make study of employment potential, location and size of labor force, et cetera.

C. Analyze the data collected under A and B above, including:
   1. Estimate of future population growth by areas within the County,
   2. Analyze the present traffic flow within the County using origin and destination and volume data to estimate the following:
      a. Average daily traffic for autos, trucks and other vehicles
      b. Prepare zone to zone traffic movement tables
      c. Prepare desire line maps,
   3. Estimate the traffic flow within the County for 5 years and 20 years hence. Use the data for the present flow adjusted to reflect the future population growth by County areas, vehicular use per resident, et cetera,
      a. Average daily traffic for autos, trucks and other vehicles
      b. Prepare zone to zone traffic movement tables
      c. Prepare desire line maps,
   4. Estimate the traffic flow within the County when a state of saturation of the population, according to land use plans, has been reached. This is to establish right of way needs and will not be carried to the same detail as 2 and 3 above.

D. Prepare a basic highway plan of the County to include all freeways, expressways and important feeders thereto including State, County and City elements. Test roadway network as determined on basis of future traffic desires by assigning traffic on both time and distance saving bases. Evaluate impact of new highway facilities on intensity of land use in adjacent areas. Modify roadway network layout as needed and check again. Use sufficient cycles of successive approximations to obtain satisfactory final plan.
   1. This will include, based on traffic volumes 5 and 20 years hence the following:
      a. Location of elements based on
      (1) Aerial photographs and topographic surveys to be provided by the County,

   (2) Present pattern of highways, roads and streets,
   (3) Traffic desires as determined in C-3, above,
  b. Capacity of units
  c. Interconnections
  d. General plans including widths of roadways, right of way requirements, roadway cross-sections, principal drainage and other essential elements.
 2. Prepare an estimate of right of way needs for traffic volumes for conditions of ultimate population saturation.
 3. Develop design standards for various components of final trafficways plan.

E. Prepare general cost estimates of the County highway plan for 5 and 20 years hence to be divided by principal structures and logical sections of highway, with items such as right of way, construction, engineering and contingencies shown separately. Costs will be based on present cost levels. The County will provide all right of way estimates.

F. Prepare a table of improvements, with priorities for those projects recommended to be constructed within 10 years.

G. Prepare and print a report including:
 1. Description of methods of study,
 2. Results of traffic counts including table and maps,
 3. Strip maps of principal highway improvements recommended,
 4. Layouts of all principal interconnections,
 5. General maps and standards of all other improvements recommended,
 6. Methods of making cost estimates,
 7. Tabulation of all cost estimates by individual improvements,
 8. Tabulation of recommended priority groups,
 9. Report will be on bond paper, or equal, with maps and figures in appropriate colors, clear type and suitably bound; 300 copies will be provided with additional copies available at cost.

3. The County agrees to provide all necessary aerial photographs, survey information and estimates of right of way costs. It further agrees that all agencies of the County will cooperate with and assist the Engineer in every reasonable way to the end that the Engineer will obtain all necessary information and data required by him to perform the services herein provided for. The County further agrees that it will make every reasonable effort to obtain similar cooperation and assistance from the State and Cities.

4. For the services set forth in paragraph 2 above the County agrees to pay the Engineer the lump sum of One Hundred Sixty Thousand Dollars ($160,000.00). This amount shall be payable in thirteen (13) installments of Eleven Thousand Dollars ($11,000.00) each, commencing on the tenth day of September, 1957, and the tenth day of each month thereafter, and a final payment of Seventeen Thousand Dollars ($17,000.00) when all work covered by paragraph 2 of this Agreement is completed.

5. It is mutually understood and agreed upon that the Engineer shall complete the work listed in paragraph 2 within fourteen (14) months of the date of this Agreement.

6. The Engineer shall coordinate his work with the County Department of Public Works and such other County, State and City agencies as the Department shall designate and shall consult with each of these agencies at such

times and frequencies as may be necessary and appropriate. The Engineer shall make monthly progress reports of his work in writing.

7. The Engineer is employed herein to render a professional service only and any payments made to him are compensation solely for such services as he may render and recommendations he may make.

This Agreement may be terminated at any time by mutual agreement of the parties hereto.

IN WITNESS WHEREOF, said County of Contra Costa has caused this Agreement to be executed by the Chairman of the Board of Supervisors and the County Clerk and caused its official seal to be hereunto fixed, and Tudor Engineering Company has caused these presents to be executed as to the date hereinabove set forth.

CITY - COUNTY - STATE COOPERATIVE ROAD STUDY CONTRACT BETWEEN THE STATE OF CALIFORNIA AND CONTRA COSTA COUNTY

THIS AGREEMENT, is entered into by and between the COUNTY OF CONTRA COSTA, State of California, a political subdivision thereof, hereinafter referred to as "COUNTY," and the STATE OF CALIFORNIA, DEPARTMENT OF PUBLIC WORKS, DIVISION OF HIGHWAYS, hereinafter referred to as "STATE."

WHEREAS, the Legislature of the State of California at its 1957 regular session adopted Senate Concurrent Resolution No. 26 which requested the Department of Public Works to undertake a study to provide a basis for an over-all State-wide plan of freeways and expressways regardless of jurisdiction; and

WHEREAS, Senate Concurrent Resolution No. 26 requests the Department of Public Works to employ by contract or otherwise engineering consultants or other specialists for conducting the study and preparing the necessary reports; and

WHEREAS, cities and counties of the State of California are requested by Senate Concurrent Resolution No. 26 to cooperate with the Department of Public Works in the conduct of the study; and

WHEREAS, it has been found by the Department of Public Works that in some cases the most expeditious and least costly method of conducting this study is to cooperate with the cities and counties in the making of an overall survey of the cities' and counties' needs in planning for future growth and for the city or county to employ the engineering consultants and specialists to perform the services for State and for the city or county; and

WHEREAS, County has entered into an agreement with such engineering consultants and specialists to conduct a study and prepare the report thereof as required by State, a copy of which agreement is attached hereto as Exhibit "A"; and

WHEREAS, State is requested by Senate Concurrent Resolution No. 26 to defray the costs of such studies from money available for highway planning, and that State now desires to enter into an agreement with County to pay for the value of services to be performed for State:

NOW, THEREFORE, it is hereby understood and agreed between the parties hereto as follows:

I. That County will:

A. Deliver to State 10 copies of the report of the consultants which will be delivered to County pursuant to Section 2. G. of Exhibit "A" and which will also contain all that information prepared pursuant to Section 2. D. of Exhibit "A."

B. Deliver to State a report containing the information provided for in Section 2. D. of Exhibit "A" immediately upon receipt thereof by County, which in no event shall County agree to receive after June 30, 1958, without the written consent of State.

C. In the event consultants do not deliver the report to County as required by Section 2. D. of Exhibit "A" by June 30, 1958, County will pay State the cost of completing the studies and preparing the report necessary for the requirements of State.

D. Upon request of State, furnish a copy of progress report which County receives from consultants employed pursuant to Exhibit "A."

E. In no event modify its agreement with the engineering consultants or specialists employed by County pursuant to Exhibit "A" without the written consent of State, if such modification affects the provisions relating to the information to be furnished to State pursuant to this agreement.

II. That State will:

A. Pay to County one-third of the amount to be paid consultants pursuant to Section 4 of Exhibit "A," but in no event shall this exceed the amount of $53,333.34, and to be paid upon receipt by State of a report of the findings and recommendations provided for in Section 2. D. and Section 2. G. of Exhibit "A," or, in the event the agreement is terminated pursuant to the agreement contained in Exhibit "A" pay to County the reasonable value of services performed for State, but in no event shall this amount exceed that which State is required to pay if the agreement had not been terminated.

III. That this agreement constitutes the entire agreement between the parties, hereto for the services to be furnished pursuant to this agreement and that this agreement can be modified, altered or revised with the written consent of both parties hereto.

IN WITNESS WHEREOF, the parties hereto have executed this agreement this _______________ day of _______________ 1957.

SENATE CONCURRENT RESOLUTION No. 26

WHEREAS, The Legislature of California finds:

(a) Adequate, safe, and economical highway transportation is vital to the future development of the State of California.

(b) It has been amply demonstrated that properly designed and located freeways and expressways are the most economical means of providing highway adequacy and safety.

(c) California is rapidly developing individual freeways and expressways and segments thereof, but in many cases on a piecemeal basis, which program has been greatly accelerated by the enactment of the Federal-Aid Highway Act of 1956 and will be expanded considerably more if Congress carries out its stated intentions regarding apportionments of federal funds for interstate highways.

(d) There is need for the people of California and its agriculture and industry to be informed of plans for the ultimate freeway and expressway system of the entire State as nearly as such can now be determined by basic engineering studies.

(e) There is need for the establishment of a plan for such a state-wide system of freeways and expressways determined without regard to present jurisdiction over the highways, roads, and streets that might be included, in order that appropriate state, county, and city transportation plans and fiscal arrangements may be worked out and properly coordinated; now, therefore, be it

Resolved by the Senate of the State of California, the Assembly thereof concurring, As follows:

(a) The Department of Public Works is requested to undertake a study which will provide a basis for an over-all state-wide plan of freeways and expressways for the State of California, such study not to be limited to state highways and such study to locate the potential freeway and expressway routes of such a state-wide system and the necessary connections thereto as nearly as is practicable in advance of detailed engineering design of projects.

(b) The Department of Public Works is requested to employ by contract or otherwise such engineering consultants or other specialists as it deems may be needed for conduct of the study and the preparation of reports.

(c) The Department of Public Works is requested to defray the costs of the study from moneys available in the State Highway Fund for highway planning and is further requested to undertake to secure matching contributions of federal funds available for highway planning to the extent that such are available.

(d) Agencies of the State Government and cities and counties and the City and County of San Francisco are requested to cooperate with the Department of Public Works in the conduct of the study.

(e) The Speaker of the Assembly and the President pro Tempore of the Senate are directed jointly to appoint a committee of seven officials of counties and seven officials of cities to act in a technical advisory capacity to the Department of Public Works, and the department is directed to cooperate and confer with the technical advisory committee so appointed.

(f) The Speaker of the Assembly and the President pro Tempore of the Senate shall refer the subject matter of this resolution to the appropriate joint interim committee which may deal with highway transportation problems if such committee is created at this session of the Legislature, or if no such committee is created, then the subject matter shall be referred to the appropriate interim committee of each of the respective houses by the Speaker and the President pro Tempore respectively.

(g) The Department of Public Works is requested to report from time to time on the progress of its study to the appropriate interim committee or committees as designated under the provisions of the preceding paragraph and to submit its final report on the subject matter of this resolution to the appropriate committee or committees not later than September 1, 1958.

SENATE CONCURRENT RESOLUTION No. 134

WHEREAS, After full legislative consideration a program of additional financing of state highways was adopted by the Legislature in 1953 and extended in 1955; and

WHEREAS, Present taxes for highway purposes are scheduled for reduction in 1959; and

WHEREAS, The Federal-aid Highway Act of 1956 has been enacted providing California with certain additional funds for state highway construction; and

WHEREAS, Further congressional action on the federal highway program to be taken in 1958 will vitally affect the financing of California's highways; and

WHEREAS, Needs of county roads and city streets are directly related to the state highway program; and

WHEREAS, The Legislature in its 1957 Session has enacted Senate Concurrent Resolution No. 26 requesting the Department of Public Works to prepare, with the advice of a select committee of county and city officials, an engineering plan for a state-wide system of freeways and expressways, and to report its findings to a legislative interim committee by September 1, 1958; and

WHEREAS, The Legislature has need of the latest complete information on all of the foregoing matters so that it may enact legislation giving proper and balanced consideration to the problems of state highways, county roads and city streets, particularly in respect to the extent of the program needed for each system and the methods of financing those programs; now, therefore, be it

Resolved by the Senate of the State of California, the Assembly thereof concurring, As follows:

1. The Joint Interim Committee on Highway Problems is hereby created, appointed, authorized, and directed to ascertain, study, and analyze all facts relating to the following:

(a) California's highway, road, and street needs, including an apprasial of all deficiencies with respect thereto;

(b) The efficiency and effectiveness of state highway, county road, and city street expenditures, including a determination of whether unnecessary, wasteful or extravagant expenditures have been or are being made by any agency of government;

(c) Budgetary practice relating to highways, roads, and streets of the State, counties, and cities;

(d) The effectiveness and efficiency of present administrative arrangements pertaining to highways, roads, and streets;

(e) The effectiveness of present policies of the state and local agencies with respect to advance planning of highway construction and the advance acquisition of rights of way;

(f) The costs and economic effects of present policies with respect to the construction of limited access highways and metropolitan freeways;

(g) The desirability of establishing a system of freeways and expressways;

(h) Alternative methods of financing the highway, road, and street modernization programs of California;

(i) Alternative methods of taxing heavy commercial vehicles equitably in relation to taxes on other highway users, including consideration of the impact and revenues derived from the motor vehicle transportation license tax and alternatives therefor;

(j) The collection and evaluation of data which will provide an equitable basis for the allocation of funds for highway purposes among the various levels of government in this State and among the geographic areas of the State;

(k) The effect of federal aid for any class or classes of highways, roads, and streets in this State, together with the related effects on other highways, roads, and streets, including the possibility of reapportioning funds allocated for highways on which increased federal aid is available for use on other highways, roads, and streets.

2. The committee shall consist of the President pro Tempore of the Senate and six other Members of the Senate appointed by the Committee on Rules thereof. Vacancies occurring in the Senate membership of the committee shall be filled by the Senate Committee on Rules. The committee shall also consist of the Speaker of the Assembly and six other Members of the Assembly appointed by the Speaker thereof. Vacancies occurring in the Assembly membership of the committee shall be filled by the Speaker or, if there be no Speaker, by the Assembly Committee on Rules.

3. The committee is authorized to act during this session of the Legislature including any recess, and after final adjournment until the commencement of the 1959 Regular Session, with authority to file a partial report not later than the fifteenth of March, and its final report no later than the last day of that session.

4. The committee and its members shall have and exercise all of the rights, duties, and powers conferred upon investigating committees and their members by the provisions of the Joint Rules of the Senate and Assembly as they are adopted and amended from time to time, which provisions are incorporated herein and made applicable to this committee and its members.

5. The committee has the following additional powers and duties:

(a) To select a chairman and a vice chairman from its membership;

(b) To employ an executive secretary, engineers, auditors, and other personnel as it deems necessary to carry out the purposes for which it is created;

(c) To contract with such other agencies, public or private, as it deems necessary for the rendition and affording of such services, facilities, studies, and reports to the committee as will best assist it to carry out the purposes for which it is created;

(d) To cooperate with and secure the cooperation of county, city, and county, and other local law enforcement agencies in investigating any matter within the scope of this resolution and to direct the sheriff of any county to serve subpenas, orders, and other process issued by the committee;

(e) To collect by questionnaires, direct investigation, or otherwise, all such information from the owners and operators of motor vehicles on the highways, roads, and streets of this State as it deems necessary in order formulate a reasonable and equitable system of highway user taxation for all classes of motor vehicle operation and to carry out the other purposes for which it is created;

(f) To report its findings and recommendations to the Legislature and to the people from time to time and at any time not later than herein provided;

(g) To meet and act at any place within the State or within the United States;

(h) To do any and all other things necessary or convenient to enable it fully and adequately to exercise its powers, perform its duties, and accomplish the objects and purposes of this resolution.

6. The committee is authorized in its discretion to appoint an advisory council to consist of nonlegislative members, but which shall include repre-

sentatives of major state-wide public and private agencies and organizations having a primary interest in streets and highways, as may be determined and appointed by the committee, which council shall aid and advise the committee in the studies conducted by the latter in carrying out the purposes of this resolution. The advisory council shall have and exercise such powers and duties as shall be defined from time to time by the committee within the scope of the powers and duties committed to the latter by this resolution.

7. The sum of fifty thousand dollars ($50,000) or so much thereof as may be necessary is hereby made available from the Contingent Funds of the Senate and Assembly for the expenses of the committee and its members, and for any charges, expenses or claims it may incur under this resolution, to be paid from said contingent funds equally and disbursed, after certification by the chairman of the committee, upon warrants drawn by the State Controller upon the State Treasurer.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3199

BANK PROTECTION ON CENTRAL VALLEY STREAMS

By R. W. Barsdale,[1] M. ASCE

SYNOPSIS

Bank protection is an important adjunct to flood control and navigation projects in the Central Valley of California. The corrective measures that have generally proven satisfactory for the various types of channels varying from tidal waters to foothill streams are described.

A brief description of the West's most distinctive topographic feature will be of convenient help in understanding the bank protection problems in the Central Valley region. Fig. 1 presents a general map of the Central Valley. The Valley is approximately 400 miles long and from 30 to 60 miles wide. It is relatively flat, varying from 10 ft below sea level in the delta region to about elevation 500 at its extremities. It is bordered on the east by the Sierra Nevada and Cascade Ranges, and on the west by the California Coast Ranges. It is drained by two main river systems, the Sacramento, having its origin in the mountains to the north, and the San Joaquin rising in the southeast. The Sacramento River flows south along the central axis of the valley, meeting the northward flowing San Joaquin River in a delta area about 30 miles west of Stockton. The joined rivers then flow west to San Francisco Bay and discharge into the Pacific Ocean through the Golden Gate.

Both of these river systems are mature, and before settlement of the valley began in 1850, periodically overflowed their banks into flanking overflow basins inundating wide areas of low lying lands. The settlers, who followed

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[1] Superv. Civ. Engr., Sacramento Dist., Corps of Engrs., Sacramento, Calif.; formerly Chf. of Levees and Channel Sect., U. S. Army Engr. Dist., Sacramento, Calif.

the miners, found that wealth could be acquired by means other than the shovel and gold pan, and valuable agricultural products rewarded their efforts, even though crops were sometimes damaged or destroyed by flood. In order to reduce this loss, the settlers began a crude system of levees to afford partial protection from the frequent floods.

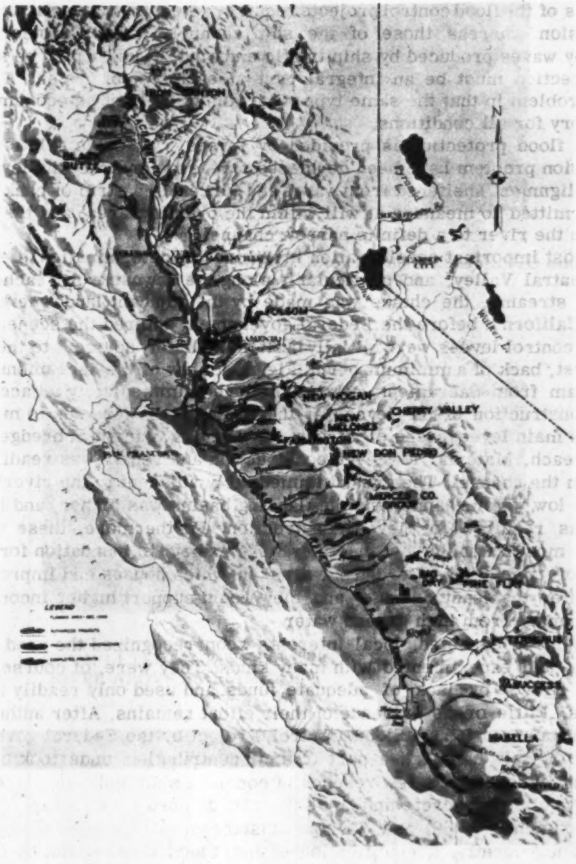
IRON CANYON
BLACK BUTTE
N
FOLSOM
NEW HOGAN
CHERRY VALLEY
NEW MELONES
FARMINGTON
NEW DON PEDRO
SAN FRANCISCO
MERCED CO. GROUP
BIG DRY
PINE FLAT
TERMINUS
SUCCESS
ISABELLA
LEGEND

FIG. 1.—CENTRAL VALLEY

This crude levee system, through local, state, and Federal construction, has expanded during the ensuing 100 yr into a very extensive and complex system of flood control, the scope of which is not widely known. There are about 1,100 miles of both project and non-project levees along the Sacramento River system, 2,400 miles along the San Joaquin River system above the delta, and 1,300 miles in the delta area, or a total of 4,800 miles of levee in the

Central Valley. Of this total, about 33%, or 1,600 miles of levee are a part of federally authorized flood control projects. In addition to these flood control projects, there are two federally authorized ship channel projects, the Stockton Ship Channel, extending 96 miles east from the Golden Gate to Stockton, and the Sacramento River Ship Channel, that will extend 43 miles north from the Stockton Channel in Suisun Bay to the vicinity of Sacramento. The banks and levees of the flood control projects require protection primarily from current erosion whereas those of the ship channels require protection from erosion by waves produced by ship traffic and wind. Thus, it may be seen that bank protection must be an integral part of each type of project. This presents a problem in that the same type of treatment will not be economical and satisfactory for all conditions.

Where flood protection is provided by levees, two basic solutions to the bank erosion problem have been developed. One is to construct the levees on a setback alignment sheltered from waves and current by a strip of bank with the river permitted to meander at will within the overflow area, and the other is to confine the river to a definite narrow channel.

The most important consideration in making this determination is economy. In the Central Valley, and particularly along the downstream reaches of the principal streams, the choice was made by the adjacent landowners and the State of California before the Federal government entered the scene. Most of the flood control levees were initially built immediately adjacent to the stream or, at most, back of a minimum berm. Fig. 2, a view of the Sacramento River, downstream from Sacramento, shows the levees immediately adjacent to the river. Construction of the levees at these locations were due to many reasons. The main levees were constructed by floating clamshell dredges having limited reach. Material for levee construction and repair was readily available from the channel. The ground immediately adjacent to the riverbank between the low water channel and the flanking basins was higher, and less material was required for levee construction. Furthermore, these rimlands were the most valuable; because they were subject to inundation for shorter periods of time, they afforded an area on which houses and improvements would suffer the least damage, and they would support higher income crops without damage from high ground water.

As might be expected, local interests soon recognized the need for bank protection and experimented with many kinds. They were, of course, limited in their efforts by lack of adequate funds and used only readily available materials. Little or no evidence of their effort remains. After authorization of the Sacramento River Flood Control Project by the Federal government, the State of California, as a part of their contribution, undertook bank protection and constructed a few reaches of concrete slab, built-in-place, one or two of articulated concrete mat, and several of dumped rock riprap. These are still in existence (1960) and, in some instances, still provide adequate protection. Subsequently, a modification of the Flood Control Act by Congress made bank protection a responsibility of the Federal government, and little bank protection has been placed by either local interests or the state since that time.

Some of the first bank protection placed by the Sacramento District of the U. S. Army, Corps of Engineers consisted of quarry stone dumped on the sloped underwater bank with fitted, hand placed, quarry stone above the low water line. Because of the high labor costs involved in hand placing the quarry stone, the woven lumber mattress, that was initially developed by the Corps

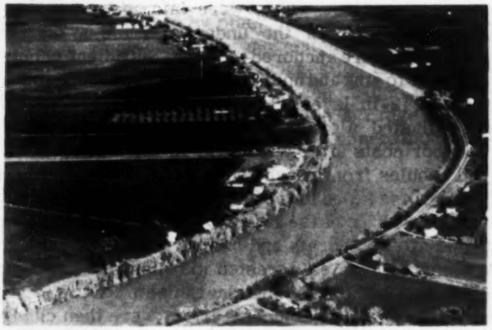

FIG. 2.—SACRAMENTO RIVER DOWNSTREAM FROM SACRAMENTO SHOWING LEVEE IMMEDIATELY ADJACENT TO RIVER

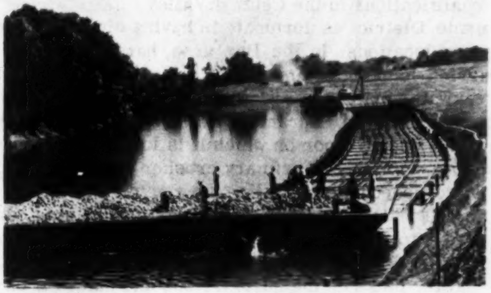

FIG. 3.—COMPLETED MATTRESS BEING BALLASTED SACRAMENTO RIVER ABOVE SACRAMENTO

FIG. 4.—COMMERCIAL ROCK PROCESSING PLANT BAY REGION

of Engineers for use along the Missouri and Mississippi Rivers, was used. These mattresses, of variable width, were laid on the prepared underwater slope from 10 ft riverward of the underwater toe up the slope to the low water line at which they were anchored to piling. Sufficient cobble rock ballast was placed on the mat to sink and hold it in place. The bank above the mat was sloped and received a 12-in. layer of graded cobbles. Fig. 3 shows a completed mattress being ballasted along the Sacramento River above Sacramento.

Due to rising labor costs after World War II, the mat could no longer compete with dumped cobbles from nearby sources. It was also found in some instances that the mattress failed in deep water due to irregularity of the underwaterslope causing breakage of the lumber, and failure in shallow water due to floating of the mat after tug boats and barges had dislodged the ballast rock. Quarry rock producers became interested in bank protection, particularily in the delta and the lower San Joaquin and Sacramento Rivers, and asked for the opportunity to bid. The plans and specifications were then changed to provide for alternate bids of either dumped quarry stone or cobbles.

Engineers, of course, recognize that suitable material for bank protection must be durable. In addition, the material should be available in quantity, be economical, reasonably easy to place, and able to resist the applied forces. It is generally agreed that rock is the most common and suitable material meeting these qualifications in the Central Valley.

The Sacramento District is fortunate in having almost unlimited supplies of rock at nearby locations. In the Bay area, hard durable sandstone ledge rock from quarries is being commercially mined and processed for highway use and for breakwater material. Fig. 4 shows the plant that produces most of the quarry rock used in the Sacramento District. The ledge rock is drilled and shot down to the quarry floor on which it is loaded by a power shovel into trucks, hauled to a 6 ft by 5 ft primary crusher and reduced to a maximum size of 14 in. The crushed material falls onto a conveyor and is carried to the screening plant. All material retained on the 5-in. screen is carried by conveyor and dropped onto 1,200 ton to 1,500 ton barges that are then towed to the site of the work. Here a whirly, or crane, equipped with a clamshell bucket mounted on a barge, picks up the quarry stone and places it on the prepared slope.

At the foothill line along the Sierra Nevada, most of the streams tributary to the Sacramento, Feather, and San Joaquin Rivers have large deposits of cobbles in the form of tailing piles left by the gold dredging operations. Fig. 5 is an aerial view of tailing piles resulting from gold dredging operations. These piles are the source of the cobbles used. The tailing material is run over a grizzly, and all material passing a 12-in. screen and retained on the 1-1/2 in. screen is loaded by clamshell into trucks and hauled to the site. The material is dumped as near to the prepared bank as possible. From here, it is picked up by clamshell and placed on the slope.

Due to the many miles of existing levee within the limits of the authorized projects and to the relatively high cost of bank protection, it has been the practice to place bank protection only at locations in which erosion is active and where it actually threatens the safety of the authorized levee, or navigation works (or combinations of the two), as opposed to placing it at all locations at which erosion might occur. Most of the bank protection that has been placed by the Federal government is in the delta of the Sacramento and San Joaquin Rivers, and on the lower and middle reaches of the major streams in which the mean channel velocities are less than 10 fps.

Bank protection standards in use in the Sacramento District are the result of experience gained since about 1936. These standards compare very favorably with standards in use by other district offices of the Corps of Engineers. The Chief of Engineers, has suggested criteria for slope protection for dams, levees, and channels. This suggestion is based on investigational projects conducted by various district offices of the Corps of Engineers between 1948 and 1952. The memorandum gave a range of velocities with the corresponding riprap thickness, maximum size, approximate grading, and filter thickness for each, and stated that the velocity to be used in selecting riprap values is that measured within 10 ft of the bank rather than the average stream velocity in the channel. Since actual bank velocity data of these kind are rarely available at erosion sites, use of this memorandum requires that correlation be made between the commonly available mean channel velocity and that which may occur 10 ft from the bank. Detailed measurements of extreme velocity distributions against the outside bank of typical bends on the Sacramento and San Joaquin Rivers have been underway since 1956, and preliminary correla-

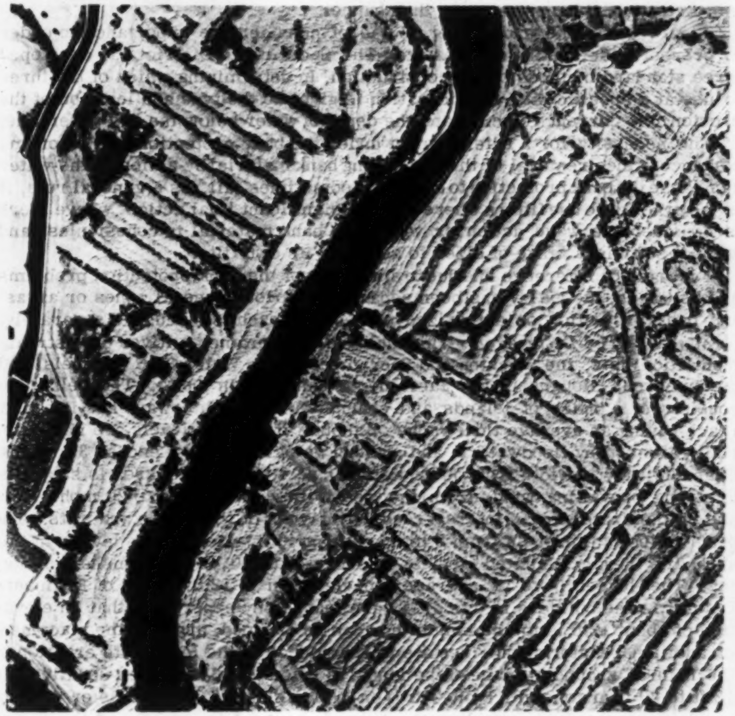

FIG. 5.—AERIAL VIEW OF TAILING PILES

tion charts are being prepared. After sufficient data have been obtained, final charts will be developed for future use.

Bank erosion, due to current action, occurs most frequently on the outside of the bends and is usually more severe where the current and thalweg (line of deepest depth) are immediately adjacent to the bank. Unless repaired, the eroded areas increase in length by extending downstream. Thus, it is important that the bank protection be extended downstream far enough to prevent erosion of the bank beyond the end of the rock. This is not necessarily at the end of the curve, but rather well past the point at which the thalweg has left the bank and begun to cross the channel.

Treatment of the underwater toe is of considerable importance in order to prevent undermining and failure. The Sacramento District has developed three standard methods of toe construction. In determining which of the three to use at a specific site, consideration is given to the type and location of the erosion, the current velocity, and the depth of water below the low water line.

The necessity for a filter blanket under the bank protection is also of importance, since removal of the supporting bank material by action of the water can cause the bank protection to settle and sometimes fail. As a general rule, it is the practice to use filters where there is significant tidal action, where there is considerable wave action or where the bank material is cohesionless and without sufficient range of sizes to serve as its own filter.

For ease of reference and due to similarity of the bank protection problems and solutions, the valley has been separated into four general zones or areas, and bank protection on authorized projects in each is discussed separately.

Zone A - The Delta Area.—This includes the Sacramento River downstream from Rio Vista, the Stockton and Sacramento Deep Water Ship Channels, and the numerous sloughs in the Sacramento-San Joaquin Delta. The Delta area consists of a maze of islands and channels. Delta waterways are in a tidal reach having an average tidal range of about 4 ft. Water levels are not materially affected by floodwaters, and channel velocities do not exceed 3 to 4 fps. Channels vary in width from 500 to as much as 4,500 ft. The area is subjected to strong prevailing winds from the west with velocities up to 20 mph. Soils are predominately peat interspersed with layers of fine sands and silts. The peat varies in thickness from 0 to more than 40 ft. Most of the levees in this area are the accumulated results of more than 50 yr of clamshell dredging. The levees have an average top elevation of 12.0, Corps of Engineers Datum. Levee builders in this area have learned by experience that an elevation of about 12.0 is the maximum that can be attained, and that the placement of additional material only results in subsidence of the levee with eruption of the natural ground at or near the landside levee toe. The levees usually have 12 ft to 20 ft crowns with 1-on-2 to 1-on-3 side slopes. The islands or polders inside the levees are subsiding at a rate of about 0.3 ft per yr, and the center of many are now more than 10 ft below sea level.

Bank erosion in this zone is caused by wave wash induced by a combination of navigation traffic and wind. A strong wind across a wide section during an extreme high tide (elevation 10.0) has been known to cause waves to break over the top of the levees. Fig. 6 shows a typical delta levee that has been severely eroded by wave action.

Cross sections and soundings through an eroded reach invariably disclose a flat beach on the waterside at about the low tide line. The toe of the caving bank is at or near elevation 5.0 that is about mean tide level. The top

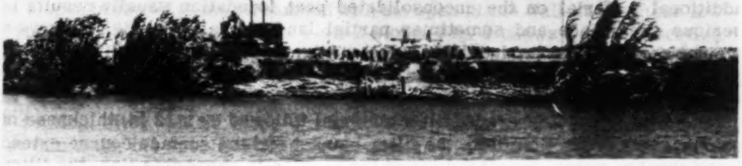

FIG. 6.—TYPICAL DELTA LEVEE SHOWING EROSION BY WAVE ACTION

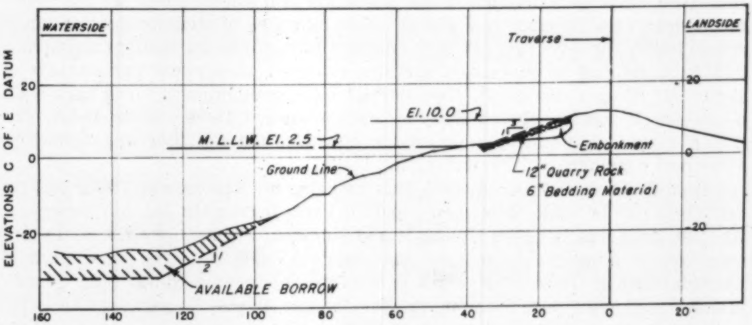
WATERSIDE
LANDSIDE
ELEVATIONS C OF E DATUM
Traverse
El. 10.0
1
2
M.L.L.W. El. 2.5
Ground Line
Embankment
12" Quarry Rock
6" Bedding Material
1
2
AVAILABLE BORROW
20
0
-20
20
0
-20
160
140
120
100
80
60
40
20
0
20

FIG. 7.—TYPICAL SECTION OF AN ERODED DELTA LEVEE AND METHOD OF REPAIR ZONE A

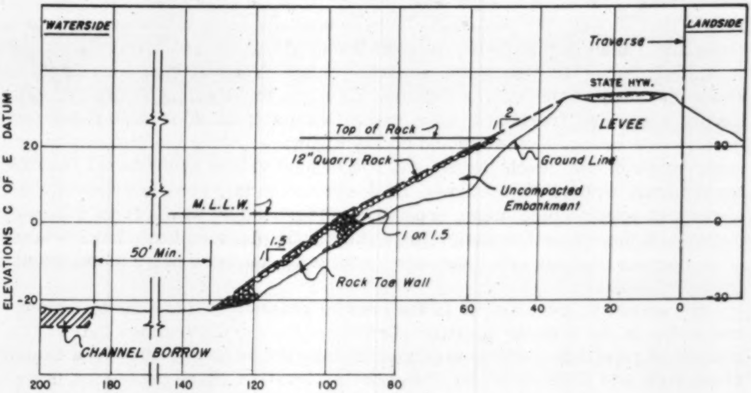
WATERSIDE
LANDSIDE
ELEVATIONS C OF E DATUM
Traverse
STATE HYW.
LEVEE
1
2
Top of Rock
12" Quarry Rock
Ground Line
Uncompacted Embankment
M.L.L.W.
1 on 1.5
1
1.5
50' Min.
Rock Toe Wall
CHANNEL BORROW
20
0
-20
20
0
-20
200
180
140
120
100
80
60
40
20
0

FIG. 8.—TYPICAL SECTION OF ERODED LOWER RIVER LEVEE SHOWING ONE METHOD OF UNDERWATER TOE TREATMENT ZONE B

of the caving bank usually terminates either on the waterside levee slope or in the levee crown. The normal procedure of sloping the riverbank and/or levee from the toe of the caving bank and adding material to the landside of the levee to get the required section is not followed, as the placement of additional material on the unconsolidated peat foundation usually results in serious subsidence and sometimes partial landside failure. Fig. 7 shows a typical section of an eroded delta levee and the method of repair. The current practice is to restore the riverside levee slope with sandy material taken from channel borrow by a floating clamshell dredge. The dressed slope is given a 6 in. blanket of graded filter material followed by a 12 in. thickness of quarry stone protection. Both the filter blanket and the surface course extend from the low water line. If the toe of the erosion is above this line, the filter and surface course are keyed into a shallow trench excavated into the beach or underwater berm. The stone protection is carried up the slope to elevation 10.0, that is 3.0 ft above the elevation of mean high tide. In this zone, the Sacramento District has placed about 52 miles of stone protection since about 1936. The average per foot contract cost of placing bank protection in this zone (including necessary earthwork) based on current price levels is about $15.00 per lineal ft. Two partial bank protection failures have been observed. It was concluded that, in one instance, failure was caused by extreme wave action during an unusually high tide, and the other was caused by abnormal subsidence of the underlying levee.

Zone B - Lower River Area.—This includes the Sacramento River and its distributaries between Sacramento and Rio Vista and the San Joaquin River and its distributaries between Mossdale and Stockton Channel. Waterways in this zone extend from the Delta area, previously described, to the upper limit of the tidal effect. This upper limit is the City of Sacramento along the Sacramento River and Mossdale along the San Joaquin River. Throughout this zone, levees have approximately 1-on-2 riverside and landside slopes and are built immediately on the riverbank or behind minimum berms. The locally built levees were constructed mainly by clamshell dredges; the soils vary from silts to clays. Mean stream velocities in the channels in this area are similar varying from 4 fps to 6 fps. Maintenance practices have been poor with the result that the narrow berms and levees generally support a heavy growth of trees and brush. The 12 ft-crown-width levees along the San Joaquin River and it distributaries in this zone generally have freeboards in excess of 5 ft above the project design flood plane. Those on the Sacramento River, while having a similar freeboard, have crown widths from 20 ft to 30 ft and frequently are occupied by state and county highways. Along the streams in this zone, most of the ranch homes and the highest valued agricultural land are immediately landside of the levee. Bank erosion in this zone is primarily due to current action. Some minor erosion may be caused by waves from a limited number of commercial vessels. Since about 1955 pleasure boats have become so numerous that an increased rate of erosion has been noted along the unprotected banks.

For economic reasons, it is the present practice in this zone to maintain the levee at its existing location. Cross sections and soundings indicate the method of repair. If a berm is in existence, the riverbank is sloped on a 1-on-2 slope from the underwater toe to the berm elevation. Stone protection is then placed from a point 10 ft riverward of the underwater toe up the slope to the berm elevation. Where the berm is narrow, or nonexistent, a 1-on-2 slope is projected riverward from the levee crown. If this slope falls within existing ground, the excess material is removed and spoiled and the stone protection is placed from 10 ft beyond the underwater toe up the prepared slope to the elevation of sustained high water during intermediate flood stages. If the slope daylights in the channel, a triangular section rock toe wall with 1-on-1-1/2 side slopes is placed on the streambed with the apex of the wall at the mean low water level and on the projected 1-on-2 slope. Fig. 8 shows a typical section of an eroded levee and one method of underwater toe treatment. Sand from channel borrow is placed behind the toe wall to the water levle. The levee or bank slope, (or a combination of them) is then brought to the desired theoretical section after which stone protection is placed on the prepared slope.

In this zone, the specifications set up alternative bids for the use of either quarry stone or cobbles. However, in the past 10 yr quarry stone, being cheaper, has been placed almost exclusively. Fig. 9 shows typical river placement of the quarry stone protection. In this zone, the Sacramento District has placed about 62 miles of bank protection at an average contract cost of about $30 per lineal foot. There have been no failures of bank protection placed under present standards. There have been, however, 2 or 3 partial failures of bank protection in which the underwater bank was protected by the woven lumber mattress previously described.

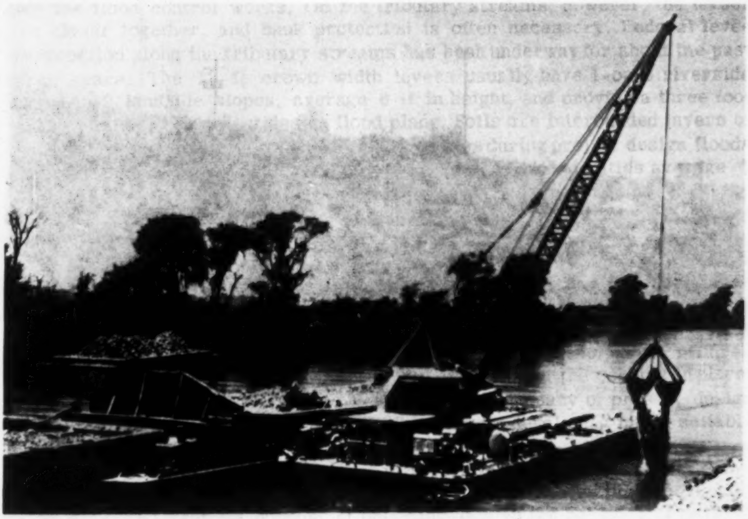

FIG. 9.—TYPICAL RIVER PLACEMENT OF THE QUARRY STONE PROTECTION ZONE B

Zone C - The Upper River Area.—This includes the Sacramento River between Colusa and Sacramento and the San Joaquin River between the Stanislaus River and Mossdale. Generally, the levees in this zone are separated from the channel by berms varying from 40 ft to about 200 ft in width. The levees, that average about 20 ft in height have at least 20 ft crown widths, 1-on-2 landside and 1-on-3 riverside slopes and have a freeboard of 3 ft or more above the project flood plane. The soils are predominately clays with some interbedded sand stratas. Levees in this zone were initially constructed variously by land equipment, clamshell dredge, and, in one or two instances, by a combination of land equipment and suction dredge. Stream velocities during extreme floods vary from 5 fps to 9 fps. Bank erosion in this zone is due almost entirely to current action, although along the Sacramento River some may be attributed to waves from limited barge traffic and pleasure craft. Fig. 10 shows a typical section of eroded bank in this zone. Water depths along the banks of the Sacramento River vary from 8 ft, during the low water season, to 40 ft, during extreme floods, while on the San Joaquin they vary from 4 ft to 25 ft.

In providing protection in this zone, it is the current practice to slope the riverbank on a 1-on-3 slope from the underwater toe to the berm elevation. Stone protection is then placed from 10 ft channelward of the underwater toe up the slope to the berm elevation. There are some reaches in this zone in which erosion has removed most of the berm. When this condition exists, the bank is cut to a 1-on-3 slope, and the levee is reconstructed on a setback alignment behind a minimum 30 ft berm. Fig. 11 indicates a typical section of a levee in this zone and the method of repair. Specifications in this zone provide for alternate bids of either cobbles or quarry stone, and experience has proven them comparable in price and performance. If quarry stone is used, it is placed from the river by floating equipment; if cobbles are used, they are truck haudd to the site and placed by land equipment.

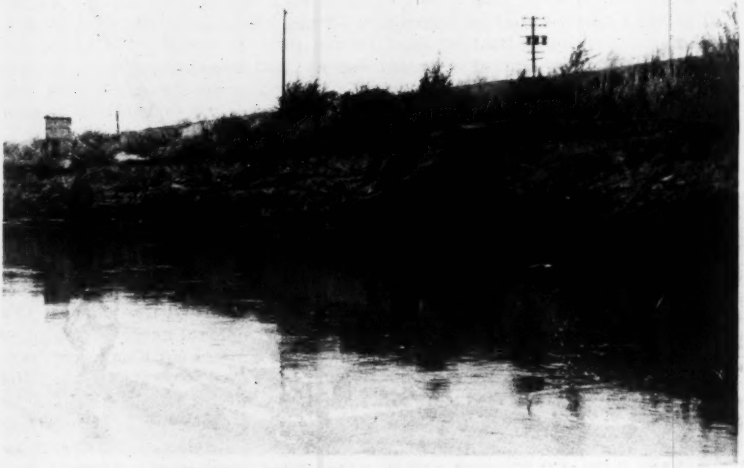

FIG. 10.—VIEW OF TYPICAL ERODED BANK IN UPPER RIVER AREA ZONE C

Both cobbles and quarry stone have proven satisfactory in this reach, and there have been no failures at locations at which current standards have been followed. However, a woven lumber mattress failed at a Sacramento River site, whereas a partial failure of cobble bank protection was experienced on the San Joaquin River in which the cobbles had been placed on a 1-on-2 slope. Fig. 12 shows a completed section of bank protection above Sacramento. The Sacramento District has placed 43 miles of bank protection in this zone at an average contract cost of about $50 per lineal foot.

Zone D - Tributary Area.—This includes the main Sacramento River upstream from Colusa, the Feather River, the San Joaquin River above the Stanislaus River, and the tributaries to all of these streams. The Sacramento District has had only limited bank protection experience in this zone. Along the upper Sacramento, San Joaquin, and Feather Rivers, the channel meanders between existing 12 ft to 20 ft crown width levees that are from 1,000 ft to more than 3,000 ft apart. Thus, bank protection is usually not necessary to protect the flood control works. On the tributary streams, however, the levees are closer together, and bank protection is often necessary. Federal levee construction along the tributary streams has been underway for about the past three years. The 12 ft crown width levees usually have 1-on-3 riverside and 1-on-2 landside slopes, average 8 ft in height, and provide a three foot freeboard over the project design flood plane. Soils are interbedded layers of clays, silts, sands, and gravels. Channel velocities during project design floods range from 10 fps to more than 12 fps, whereas overbank velocities average 5 fps. Bank protection on the tributary streams is normally required on the outside of curves, through bridges, and adjacent to irrigation and drainage structures.

The riverbank is sloped on a 1-on-3 slope from a point 5 ft below the present stream thalweg to the berm elevation. Cobbles are usually available nearby and have been specified in past construction. The Sacramento District has placed about 14.0 miles of bank protection in this zone. Although only medium flood flows have been experienced to date on one stream, two partial failures have been experienced and have been repaired by grouting the cobbles in place. Studies are currently underway to determine the adequacy of present design criteria for these tributary streams with a view to developing more suitable standards.

It might be of interest to note that in 1959, the Sacramento District completed construction of seven test sections of bank protection on the San Joaquin River in Zone C, for the purpose of obtaining data on the behavior of several kinds of untested bank protection materials when subjected to actual field conditions. The data obtained will be used in determining the most suitable and economical available materials on future work along the San Joaquin River as well as on other bank protection projects within the Sacramento District. The sites chosen for the tests are located on the left bank of the San Joaquin River just upstream from Mossdale; they are all on the outside of bends where erosion has been active. Mean channel velocities during the project design flood of 52,000 cfs are expected to reach 7.0 fps. The underlying soils consist predominatly of stratified layers of silty sand and sandy silt with an interbed of sandy clays. At each location, the levee was set back to provide at least a minimum berm of 30 ft between the top of the sloped riverbank and the river-

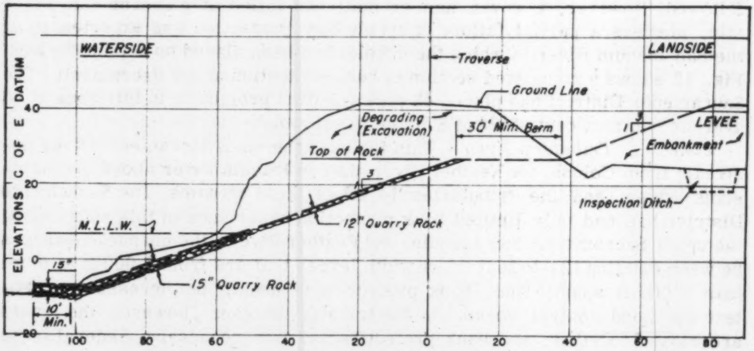
WATERSIDE
LANDSIDE
ELEVATIONS C OF E DATUM
Traverse
Ground Line
Degrading (Excavation)
30' Min. Berm
LEVEE
Embankment
Top of Rock
Inspection Ditch
12" Quarry Rock
M. L. L. W.
15" Quarry Rock
15"
10' Min.
40
20
0
-20
100
80
60
40
20
0
20
40
60
80
1
3

FIG. 11.—TYPICAL SECTION OF ERODED LEVEE ON UPPER RIVER AREA AND METHOD OF REPAIR ZONE C

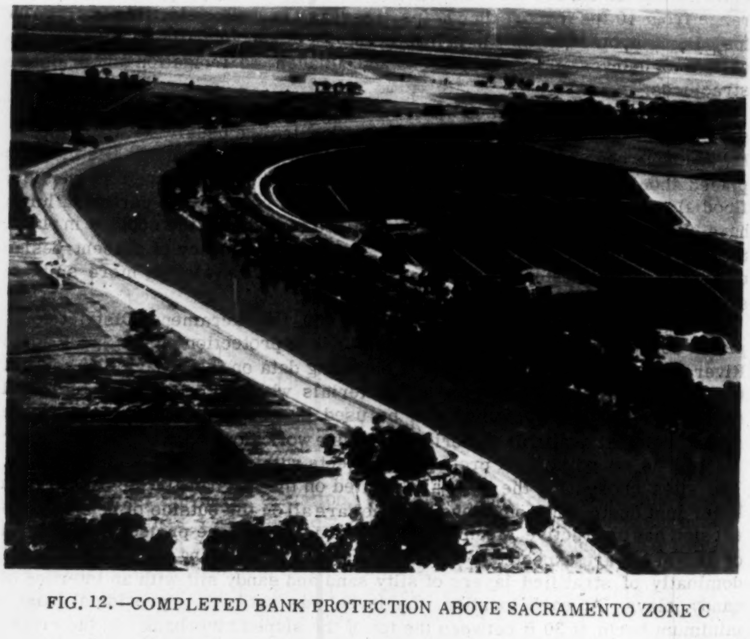

FIG. 12.—COMPLETED BANK PROTECTION ABOVE SACRAMENTO ZONE C

ward toe of the levee. The natural riverbank was excavated on a 1-on-3 slope from the bed of the river to the berm level at natural gorund.

Material being tested includes (1) graded cobbles on a filter blanket, (2) graded cobbles on natural ground, (3) bank run gravels, (4) processed gravels, (5) bank run dredge tailings, (6) compacted asphalt on a filter blanket above water with quarry stone below, and (7) uncompacted asphalt on natural ground above water with stone below. Fig. 13 shows the completed uncompacted asphalt bank protection. Initial channel cross sections have been secured at each

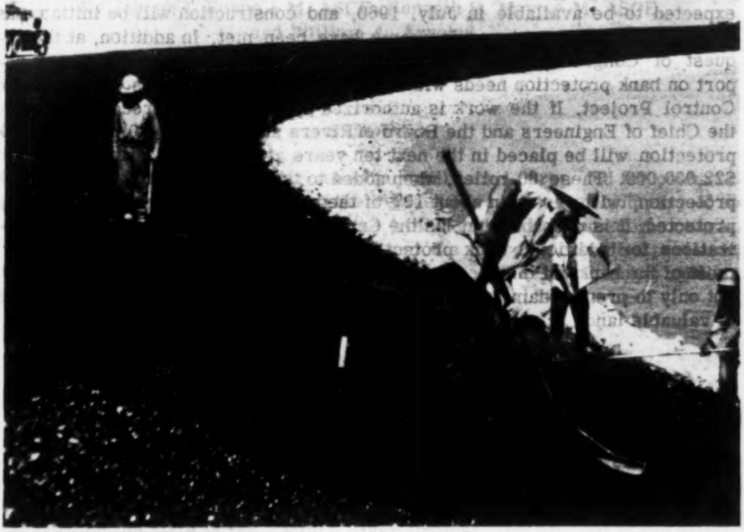

FIG. 13.—TEST SECTION ON SAN JOAQUIN RIVER UNCOMPACTED ASPHALT BANK PROTECTION ZONE D

site and will be retaken after major floods. Bank velocity measurements by current meter will be obtained 10 ft from the bank at each site over a range of flows between 10,000 cfs, and the project design flow. At the time that these measurements are taken, the mean velocity in the section will be obtained for correlation purposes. To date, there has been no high water on the San Joaquin River, and the sections are untested. All sites will be inspected annually as well as after reach high water stage when flows exceed 10,000 cfs.

One important factor affecting completed bank protection is maintenance. After a job is completed, local interests usually take the attitude that a permanent solution to the problem has been provided. Whether this is due to the excellence of design and construction or ignorance of the forces of nature, is a matter for conjecture. It is the experience in the Central Valley that bank protection maintenance is hard to "sell" local interests. It is recognized that the problem is one of education, and we are increasing our efforts in that direction.

CONCLUSIONS

Local interests recognize the need for and the importance of bank protection in the Central Valley, and particularly along the Sacramento River. Through Congressional action in 1958, they secured authorization of about 6 miles of bank protection in the upper Sacramento River, upstream from the flood control project, at an estimated cost of $1,500,000. Design funds are expected to be available in July, 1960, and construction will be initiated as soon as local interest requirements have been met. In addition, at the request of Congress, the Corps of Engineers, 1959, completed a survey report on bank protection needs within the limits of the Sacramento River Flood Control Project. If the work is authorized by Congress, as recommended by the Chief of Engineers and the Board of Rivers and Harbors, 80 miles of bank protection will be placed in the next ten years at an overall estimated cost of $22,000,000. These 80 miles, when added to the 171 miles of completed bank protection, will result in about 16% of the banks of authorized projects being protected. It is probable that, as the Central Valley develops, further authorizations for additional bank protection will be forthcoming until, eventually, most of the banks of the Sacramento and San Joaquin Rivers will be protected, not only to prevent damage to the flood control works, but to also save the loss of valuable land.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3201

HYDROLOGY AND FLOOD CONTROL FEATURES OF OROVILLE DAM

By Joseph I. Burns,[1] M. ASCE, Kenneth B. Mayo,[2] M. ASCE, and William A. Arvola[3]

SYNOPSIS

Oroville Dam, a 735-ft high, zoned earthfill dam, is proposed for construction by the State of California on the Feather River, a major tributary of the Sacramento River. A summary of the hydrologic and meteorologic characteristics of the Feather River Basin and the hydrologic studies pertaining to the design floods for the project are presented. The preliminary spillway design concepts and proposed operating criteria are also examined.

INTRODUCTION

The State of California has embarked on a vast water development program, termed the California Water Plan, to conserve and use the valuable water resources of the State. The initial unit of the California Water Plan to be built by the State is the Feather River Project. The key unit of the project is a zoned earthfill dam, 735 ft high, to be built on the main stem of the Feather River near the City of Oroville and approximately 70 miles north of Sacramento. The reservoir behind the dam will impound 3,523,000 acre-ft of

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water. The multi-purpose structure will not only provide conservation storage but will provide urgently needed flood protection to a highly developed flood plain.

Presented in this paper are the hydrologic and meteorologic characteristics of the Feather River Basin, the design floods developed for the project, the preliminary spillway plans for Oroville Dam, and the steps being taken in anticipation of the future operation of the project.

DESCRIPTION OF FEATHER RIVER DRAINAGE SYSTEM

The Feather River drainage system is situated on the western slope of the Sierra Nevada and in the northeastern section of the Sacramento Valley. The drainage system comprises the Feather River Basin, Yuba River, and Bear River watersheds, and the valley flood plain as depicted on Fig. 1. In this paper, the term Feather River drainage system will be used for the entire watershed and the term Feather River Basin will denote the Feather River drainage area above Oroville Dam Site. The drainage areas for the total watershed are as follows:

Drainage Areas	Sq Miles
Feather River at Oroville Dam Site	3,611
Feather River above Confluence of Yuba River	3,978
Yuba River at Mouth	1,350
Bear River at Mouth	550
Feather River at Mouth	5,922

Feather River Basin.—The drainage net of the Feather River Basin, delineated in Fig. 2, comprises three main forks of the Feather River. These three tributaries, North Fork, Middle Fork, and the South Fork, join immediately upstream from the Oroville Dam Site. The North Fork of the Feather River drains 2,090 sq miles or 58 % of the total basin. The eastern part of the basin is drained by the Middle and North Forks, which cut through the abrupt frontal slope of the basin in deep, precipitous canyons. Elevations in the basin range from approximately 200 ft near Oroville to 10,400 ft at Mount Lassen. However, only limited and isolated areas of the Feather River Basin are above the 7,000-ft contour and more than 50% of the area is in the 5,000-ft to 7,000-ft zone. The crest line of the frontal slope, designated on Fig. 2 as top of windward slope, averages 6,000 ft in elevation. The effect of the topography on the natural drainage is to create streams of moderate gradient in the upper portion of the basin. The gradients become quite steep, as high as 150 ft per mile, as the streams flow westerly through the deep, rugged canyons on the frontal slope of the basin.

The natural features of the basin are characterized by extensive areas of merchantable timber land, extremely rough and rocky terrain cut by precipitous canyons, and isolated mountain valleys. At the lower elevations, dense oak groves and low brush are the dominant vegetative cover. Above an elevation of 1,500 ft to 2,000 ft, the oaks merge with coniferous forests. These forests of pine, fir, and cedar are broken by outcrops of granite, deep and rugged canyons, and mountain valleys. The cover of the eastern portion of the watershed is characterized by the growth of less dense coniferous forests

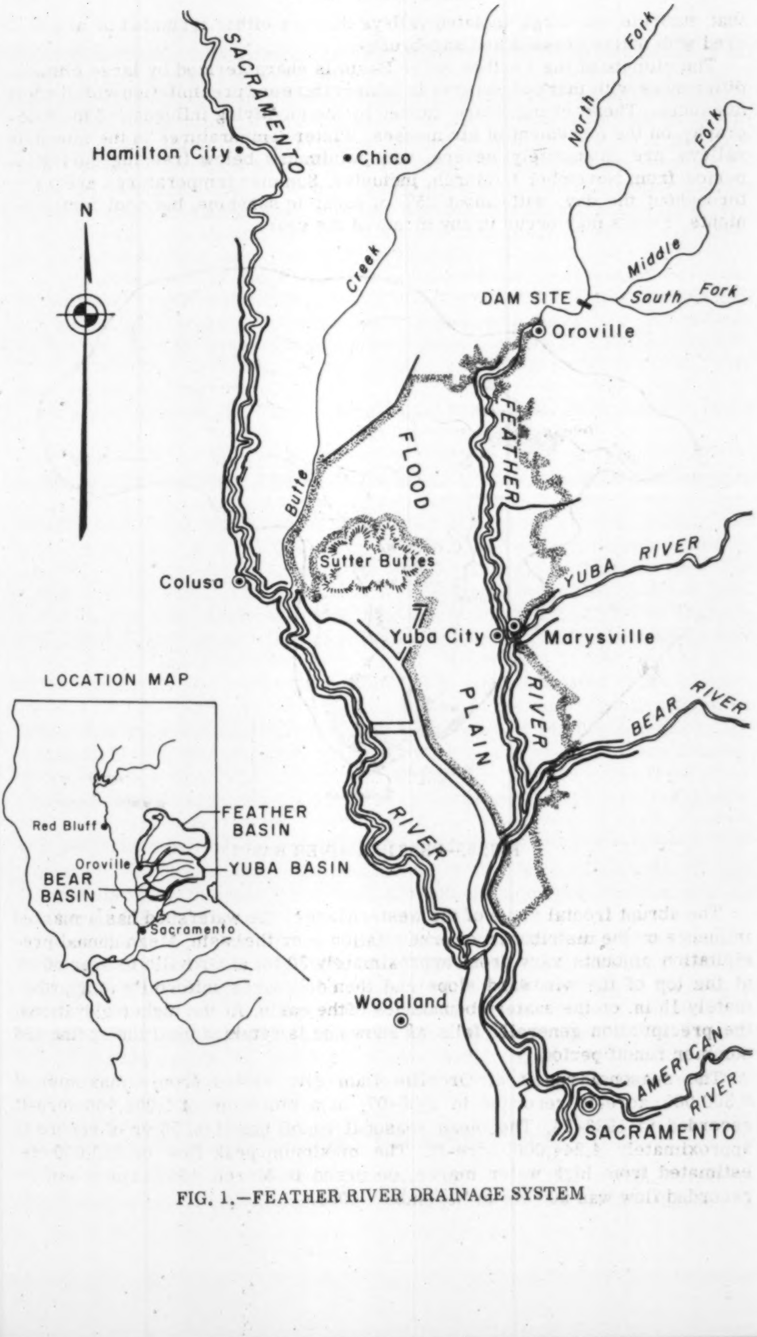
SACRAMENTO
Hamilton City
Chico
North Fork
Middle Fork
South Fork
DAM SITE
Oroville
N
Butte Creek
FEATHER RIVER
FLOOD PLAIN
Sutter Buttes
Colusa
YUBA RIVER
Yuba City
Marysville
BEAR RIVER
RIVER
Woodland
AMERICAN RIVER
SACRAMENTO
LOCATION MAP
Red Bluff
FEATHER BASIN
Oroville
YUBA BASIN
BEAR BASIN
Sacramento

FIG. 1.—FEATHER RIVER DRAINAGE SYSTEM

that encircle the large isolated valleys that are either irrigated or are covered with native grasses and sagebrush.

The climate of the Feather River Basin is characterized by large climatic differences with marked changes in temperature and precipitation within short distances. These changes are caused by the modifying influence of the topography on the movement of air masses. Winter temperatures in the mountain valleys are moderately severe, with minimums below freezing during the period from November to March, inclusive. Summer temperatures are warm throughout the day, with about 95% of possible sunshine, but cool during the nights. Frosts may occur in any month of the year.

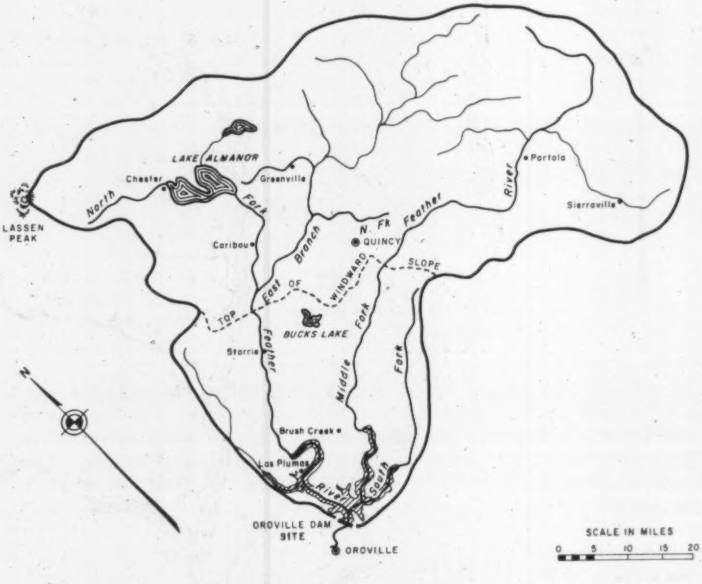
LAKE ALMANOR
Chester
Greenville
LASSEN PEAK
North Fork
Caribou
East Branch
N. Fk Feather
QUINCY
River
Portola
Sierraville
TOP OF WINDWARD SLOPE
BUCKS LAKE
Storrie
Feather
Middle Fork
Fork
N
Brush Creek
Las Plumas
River
South
OROVILLE DAM SITE
OROVILLE
SCALE IN MILES
0 5 10 15 20

FIG. 2.—FEATHER RIVER BASIN

The abrupt frontal slope of the western face of the watershed has a marked influence on the distribution of precipitation over the basin. Mean annual precipitation amounts vary from approximately 30 in. at Oroville to over 80 in. at the top of the windward slope and then decreases uniformily to approximately 15 in. on the eastern boundaries of the basin. At the higher elevations, the precipitation generally falls as snow and is retained until the spring and summer runoff period.

The seasonal runoff at Oroville Dam Site varies from a maximum of 9,300,000 acre-ft recorded in 1906-07, to a minimum of 1,200,000 acre-ft recorded in 1923-24. The mean seasonal runoff based on 55 yr of record is approximately 4,244,000 acre-ft. The maximum peak flow of 230,000 cfs, estimated from high water marks, occurred in March, 1907. The minimum recorded flow was 300 cfs in November, 1931.

The climate and runoff in the Feather River Basin are recorded by an extensive network of hydrologic stations. Twenty precipitation stations are located with the basin of which nineteen stations have an unbroken record of 10 yr durations or longer. Twelve of the stations are located between 4,000 ft and 5,000 ft elevation. There are nineteen snow courses located within the Feather River Basin that are measured and maintained as part of the California Cooperative Snow Survey program. Nine of these snow courses are located at elevations above 6,000 ft. There are eighteen stream gaging stations located within the basin. Even with this network of stations, many areas with marked changes in climate and runoff are not adequately measured to delineate the considerable variations in these factors.

The North Fork of the Feather River is extensively developed for power generation by the Pacific Gas and Electric Company. The only reservoir that has sufficient storage to exert a major influence on the flood runoff at Oroville Dam Site is Lake Almanor with a total capacity of 1,308,000 acre-ft. This reservoir completely controls approximately 500 sq miles of the North Fork drainage area.

Feather River Flood Plain.—The Feather River flood plain below Oroville Dam Site, depicted in Fig. 1, comprises a total area of 292,000 acres. The flood plain area, as defined, is artificially bounded in certain areas by levees of the Sacramento River Flood Control Project. This total flood plain area is composed of 9,000 acres of urban and sub-urban lands, 245,000 acres of agriculture lands, and 38,000 acres of land that is used for grazing, mining, and game preserves. The 245,000 acres of land used for agriculture consists of 190,000 acres of field and row crops and 55,000 acres of fruit and nuts. The current (1959) value of all lands and improvements in the flood plain is estimated by the U.S. Army Corps of Engineers at about $650,000,000 of which $275,000,000 are urban values and $375,000,000 are rural values. The cities of Marysville and Yuba City, with a total population of 21,000, are the main cities completely within the flood plain. It has been estimated by the Department of Water Resources that the population in the three counties of Butte, Sutter, and Yuba will increase from the 1959 figure of 148,000 persons to approximately 600,000 persons by 2010. Most of the areas of these counties where this growth will occur are within the Feather River flood plain.

The Feather River flood plain area is protected by 205 miles of levees. The levees are a part of the Sacramento River Flood Control Project, which was authorized by Congress in 1917. The design capacities of this levee system are 210,000 cfs above the confluence of the Feather and Yuba Rivers; 300,000 cfs below this confluence; and 320,000 cfs below the confluence of the Feather and Bear Rivers.

Since the turn of the century, major floods have occurred in the Feather River system in 1904, 1907, 1909, 1928, 1937, 1940, 1942, 1950, and 1955. In 1955, 100,000 acres of the flood plain were flooded resulting in the loss of forty lives and a total damage estimated by the Corps of Engineers at $55,581,000. This flood and breaching of project levees resulted in the evacuation of 30,000 people, flooded 3,300 homes, and drowned 6,000 head of stock. Approximately 3,000 acres of peach, walnut, and plum orchards were lost in the flooded area below Yuba City.

METEOROLOGIC CHARACTERISTICS

The storms that produce excessive amounts of rainfall to high elevations in the northern Sierra Nevada with resulting high rates of runoff are characterized by strong, sustained southwest winds. The air mass in these storms is identified as modified polar air that has acquired tropical characteristics. All the major flood-producing storms in the Feather River drainage system are essentially the same type, differing only in orientation and intensity. During November through April, such storms result from the weakening of the semipermanent Pacific high pressure center, which greatly influences California's climate, and the strengthening of the Aleutian low pressure center. In this period, storms originating on the Pacific polar front move progressively further south and pass inland through Washington, Oregon, and California. At times, the large-scale atmospheric flow pattern is characterized by a blocking high pressure center at high latitudes that diverts the storms more directly into California from the Pacific and creates a potential flood situation with warm, tropical air masses carried inland by strong winds. When such a storm track over California persists for many weeks, conditions are especially conducive for prolonged rainy periods in the state.

Precipitation in California is largely of orographic origin when strong winds carry moisture-laden air up the slopes of the Coast Range and Sierra Nevada to be released as rain or snow. Precipitation occurring within the storm centers and along the fronts also results from the horizontal convergence of moist air masses. This contribution adds to the orographic component to give the total storm precipitation.

FLOOD RUNOFF CHARACTERISTICS

The flood runoff from the frontal 1,000 sq miles of the Feather River Basin in very rapid. The occurrence of high intensity bursts of precipitation on this windward slope combined with the efficient collection of runoff by the high gradient stream system, concentrates the flow very rapidly and results in the almost "flashy" characteristics of the flood hydrographs at the Oroville Dam Site. These characteristics can best be illustrated by examining the storms and resulting runoff for two recent storm periods.

February 1960 Storm Period.—In the week preceding February 6, 1960, the mid-tropospheric flow pattern was characterized by strong westerly flow over the eastern Pacific with low pressure over the Gulf of Alaska. At the 500-millibar level (mid-troposphere), a series of short-wave troughs moved eastward and entered the Pacific Coast in Oregon, Washington, and British Columbia. On the surface weather chart, the polar front was located approximately 30°N latitude in the Pacific, and on the front a series of waves moved eastward and reached the California-Oregon-Washington coast as occluded fronts.

On February 7, 1960, an active low was located on the polar front about 2,000 nautical miles due west from San Francisco. This low deepened and moved northeastward at a speed of 35 to 40 knots. The cold front associated with this low also moved rapidly eastward and crossed the California coast in the early hours of February 8, 1960. Rain began to fall over the Feather River Basin on the 7th, and the greatest intensity fell during the night of the 7th and early hours of the 8th. During this period, the freezing level was at approximately 8,000 ft. The highest hourly rate of rain as recorded at the Brush Creek precipitation station (Fig. 2) was 0.70 in per hr, and the accumulated rainfall at this station was approximately 9 in. ,in 26 hr. The cold front passed the Feather River Basin around noon on February 8, and with its passage the heaviest precipitation terminated.

Prior to this storm, there had been only a nominal amount of precipitation on the Feather River Basin resulting in relatively dry antecedent ground conditions. At the beginning of the storm, there was a flow of only 5,000 cfs in the Feather River at Oroville Dam Site. However, the rain associated with the storm resulted in a very rapid rise of the Feather River flow from 5,000 cfs to 133,500 cfs in 24 hr. The hydrograph of runoff at the Oroville Dam Site and the hourly precipitation at Brush Creek precipitation station are depicted in Fig. 3. Note that the peak flow at Oroville occurred almost concurrently with the passage of the cold front and the abrupt cessation of effective rain at Brush Creek. It is apparent that the main contribution to the peak flow at Oroville came from the approximately 1,000 sq miles of the frontal slope area of the basin.

December 1955 Storm Period.—The weather situation in December, 1955, was characterized by an extensive blocking high pressure center located over the Bering Sea which displaced the Pacific polar front further south than normal over the Pacific Ocean. The front extended from a point just north of the Hawaiian Islands northeastward toward northern California. The confluent flow of very moist tropical air and cold air from the north Pacific along the frontal zone produced a very strong front and a very pronounced mid-tropospheric jet stream. Waves formed along the front over the ocean and moved rapidly toward northern and central California. Between the 17th and 24th of December, five waves moved into northern California bringing fove "bursts" of heavy rain lasting generally from 12 hr to 18 hr. The heaviest bursts were on the 19th and 21st. The maximum hourly precipitation rate at Brush Creek was 0.64 in. per hr on the 19th, and 0.85 in. per hr on the 21st. The total rainfall at Brush Creek in the 8-day period amounted to 30 in. The normal precipitation at this station is 69 in. Rainfall was also heavy over other parts of the Feather River Basin and the neighboring Yuba River Basin. The warm air mass carried eastward by the strong winds had a freezing level near 10,000 ft, and therefore, rain fell to high elevations in the mountains. The plot of hourly rainfall at Brush Creek during the period December 18-25, 1955, is shown on Fig. 4.

The resulting runoff, also depicted on Fig. 4, produced three major peaks of 95,000 cfs on December 19, 175,000 cfs on December 22, and 203,000 cfs on December 23. Note that the storm bursts on December 21-22 and December 23 produced approximately twice the runoff per unit precipitation that was produced by the rain of December 19. This can be attributed to the fact that the detention storage and field moisture requirements of the basin were being met and the snow line during this phase of the storm was relatively low. Note that these peak flows occurred approximately 4 hr after the intense bursts of rain at Brush Creek. By the start of the hydrograph rise on December 23, snow below elevation 6,000 ft had effectively disappeared; therefore, there was a negligible contribution from melting snow during the peak runoff. The peak flow of December 23, combined with the record runoff from the Yuba River, resulted in the levee failure below Yuba City on December 24, 1955.

DESIGN FLOODS

The design of the flood control features of the Oroville Dam is of prime importance because of not only the unprecedented height of the zoned type earthfill dam but because of the proximity of the dam to the highly developed Sacramento Valley flood plain.

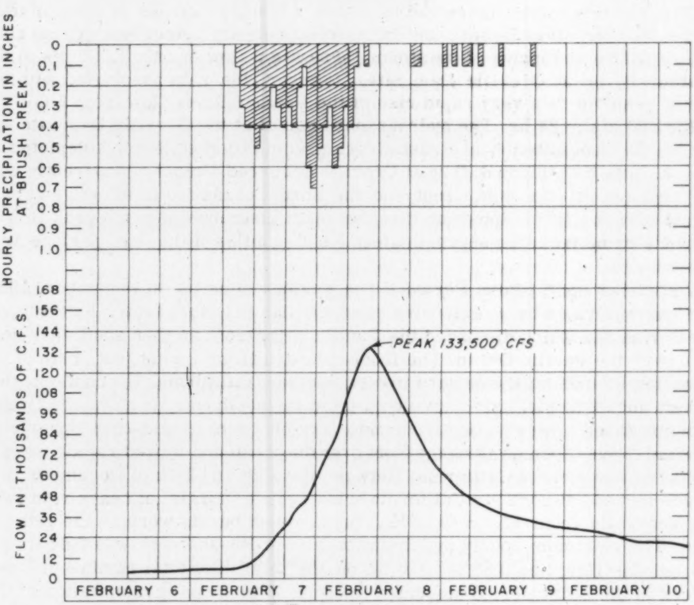

FIG. 3.—PRECIPITATION AT BRUSH CREEK AND HYDROGRAPH AT OROVILLE DAM SITE, FEBRUARY, 1960

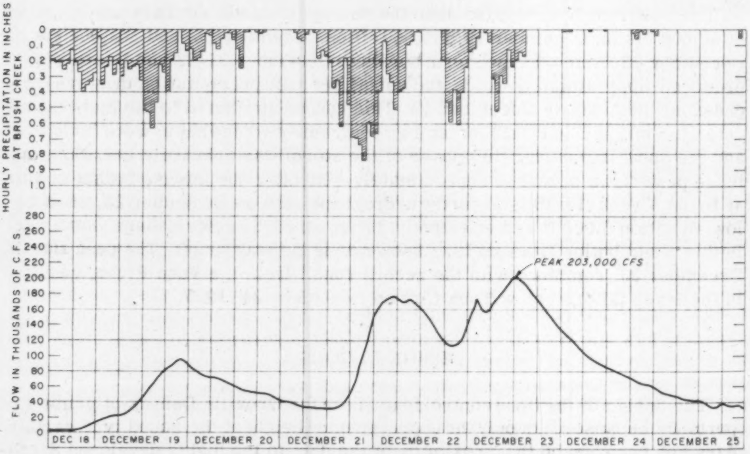

FIG. 4.—PRECIPITATION AT BRUSH CREEK AND HYDROGRAPH AT OROVILLE DAM SITE, DECEMBER, 1955

Probable Maximum Storm Studies by Department of Water Resources.—Because of the tremendous scope of the California Water Plan and the magnitude of many of the individual projects, the Department of Water Resources in 1958 retained Joseph B. Knox to derive a method for obtaining and evaluating estimates of extreme rates of precipitation on large watersheds in California. The results of these studies have been published by the Department of Water Resources[4].

The principal objective of the study was to develop a technique for estimating maximum accumulated precipitation for the Feather River Basin by applying extreme values of certain meteorological parameters to an atmospheric model. This model has provisions for including both orographic rainfall and rainfall from large-scale synoptic disturbances. These components are combined in an equation that describes the variation of the total precipitation rate with time.

The necessary computations were programmed for an electronic computer. The over-all model was tested by reproduction of precipitation in a 72-hr period during the December 1955 storm in the Feather River Basin.

Orographic Precipitation.—To determine the orographic rate of precipitation, the behavior of the wind field moving over the basin must be known. The influence of the topography upon the winds can be obtained by considering simplified terrain profiles in which the essential features of the mountain barrier are preserved. The simplified terrain profiles were obtained by smoothing the plots of the average terrain in three strips (Fig. 7 and 8), each 16 miles in width and oriented SW-NE across the Feather River Basin (an example of the profiles is depicted on Fig. 6). The initial segment of the simplified profile consists of the sloping temperature inversion layer over the Sacramento Valley. Intervening valleys within the basin are considered to be capped by level inversion surfaces over which the warm air mass is assumed to move horizontally. The inflow winds as a function of height, upwind of the mountain barrier in essentially undisturbed flow, are obtained from meteorological upper-air data. By assuming a distribution of vertical velocity with height, and using the principal of continuity, the horizontal and vertical components of the wind were computed at each 500-m level from the surface to the tropopause (approximately 9,000 m) on verticals spaced at intervals of approximately 2 miles along the profiles. An example of the vertical profiles of the horizontal inflow and outflow winds is shown on Fig. 5.

With the wind field across the basin computed, the trajectories were computed for rain drops of a given size that terminate their earthward fall at the bottom of the verticals in the computational grid. Separate trajectories for rain drops with terminal fall velocities of 4, 6, and 8 m per sec were computed. An example of the rainfall trajectories for rain drops with terminal fall velocity of 4 m per sec for the December 1955 storm, is depiected on Fig. 5 for Profile "A." (The simplified computational terrain profile is also shown on Fig. 5 for Profile "A.") Using the results of the computations of rain drop trajectories, it is possible to compute the orographic rainfall at the base of each vertical in the computational grid, assuming a saturated adiabatic atmosphere of a given moisture content.

Large-Scale Synoptic Rainfall.—The rate of precipitation due to large-scale vertical motion in the atmosphere was computed by applying an assumed vertical velocity (resulting from the dynamics of the storm circulation) to the same air mass used in the orographic computations. Care was taken to insure that the large-scale (termed "frontal") precipitation used could reasonably be expected to occur during the storm-type characterized by extreme orographic precipitation.

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[4] "Procedures for Estimating Maximum Possible Precipitation," by Joseph B. Knox, State of California, Dept. of Water Resources, Bulletin 88, Sacramento, Calif., May, 1960.

Time Distribution of Precipitation Rate.—The time distribution of precipitation rate is based on the fact that moving frontal disturbances, such as occurred during the December 1955 storms, produce a "periodic" variation in the rate of precipitation. The "periodic" rate of precipitation is quite normal as this storm type is marked by a strong southwesterly basic current in which frontal waves propagate. The maximum hourly rates of precipitation are computed from the values obtained in the orographic and frontal precipitation computations. The time-distribution model takes into account the fact that the saturation potential temperature (a measure of the moisture content of the air mass), the horizontal wind, and the frontal component of vertical motion are not independent of time.

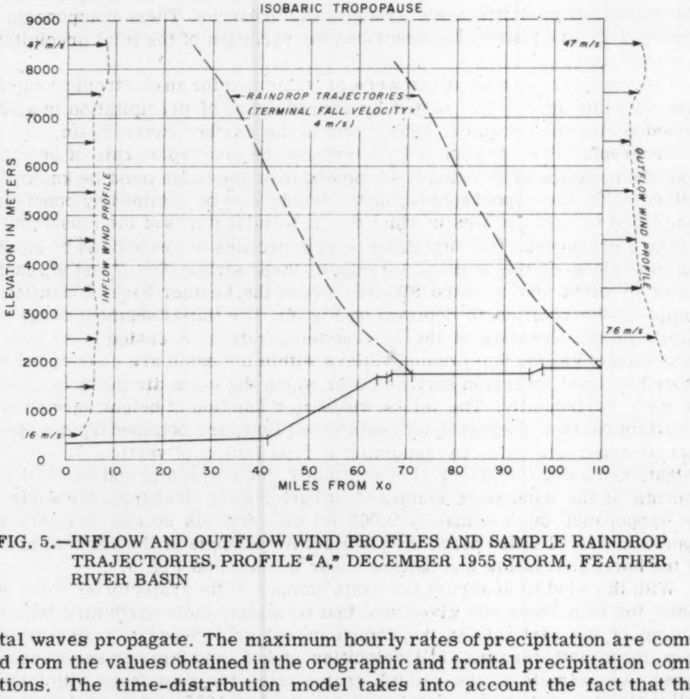

FIG. 5.—INFLOW AND OUTFLOW WIND PROFILES AND SAMPLE RAINDROP TRAJECTORIES, PROFILE "A," DECEMBER 1955 STORM, FEATHER RIVER BASIN

The instantaneous rate of precipitation is represented by an idealized sinusoidal function that closely resembles variations in recorded rates of precipitation. Critical periods were computed for this function, with a period of 59 hr being selected as the most critical for a total storm duration of 72 hr.

Nature does not produce precipitation rates that can be represented by a simple sine or cosine curve. However, Knox has shown that the total precipitation resulting from a complex time distribution of hourly precipitation is less than the total precipitation resulting from the presence of the most

adverse period alone, provided the maximum hourly precipitation in the two cases is the same. For design storms, therefore, the choice of the critical period (for a particular storm duration) represents a reasonable extreme possible in nature.

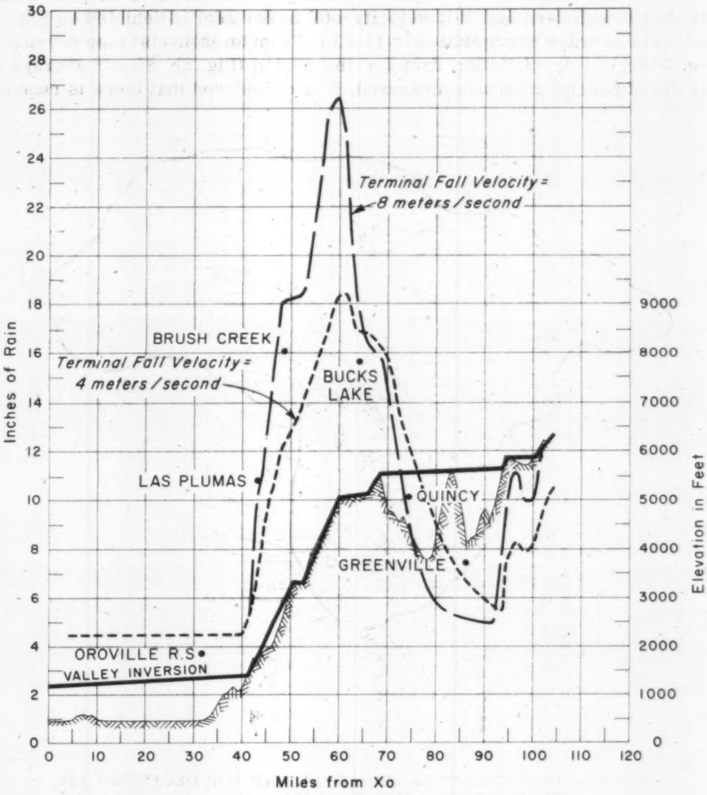

FIG. 6.—SIMPLIFIED TERRAIN PROFILE AND COMPUTED 72-HOUR RAINFALL AMOUNTS FOR DECEMBER 1955 STORM, PROFILE "B," FEATHER RIVER BASIN

Test of Model On December 1955 Storm.—Based on upper air data observations during the December 1955 storm, the parameters required for computing the orographic precipitation and frontal precipitation were obtanied. By machine computation, the accumulated 72-hr precipitation for the three assumed rain drop terminal fall velocities for the period ending 0700 Pacific Standard Time (PST) December 22, 1955, were computed for the three profile strips across the Feather River Basin. The total 72-hr precipitation amounts and

simplified terrain profile for Profile "B," for assumed rain drop terminal fall velocities of 4 and 8 m per sec are plotted on Fig. 6. For comparison, precipitation data obtained from six stations within the 16-mile wide strip for the same 72-hr period are plotted on the plate as individual points.

The corresponding computed isohyetal map for the same 72-hr period (for rain drops with terminal fall velocity of 6 m per sec) is depicted on Fig. 7. The basin average precipitation is 11.8 in. From an isohyetal map developed from observed precipitation data for the period (Fig. 8), a basin average of 10.1 in. of precipitation was measured. It is considered that there is reasonable agreement between the computed and observed precipitation values, particularly since there is a paucity of observed data in the areas of maximum precipitation.

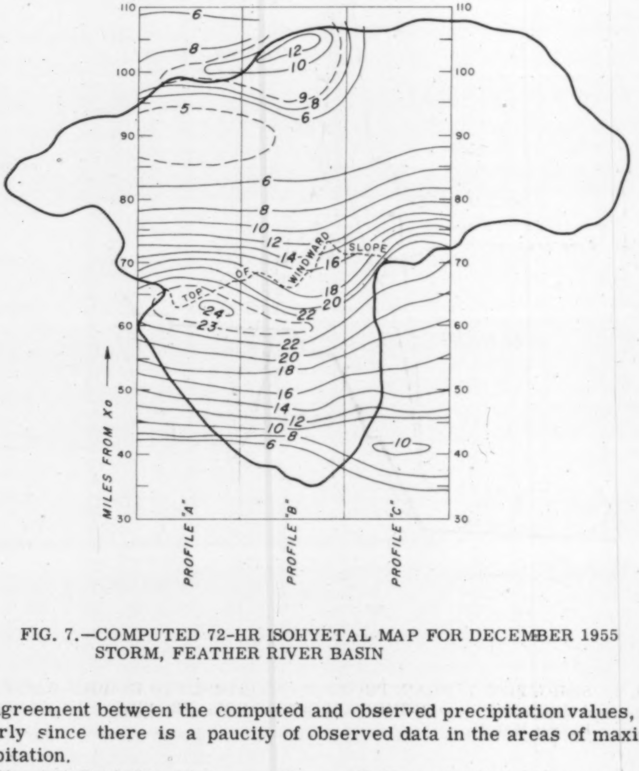

FIG. 7.—COMPUTED 72-HR ISOHYETAL MAP FOR DECEMBER 1955 STORM, FEATHER RIVER BASIN

Estimated Probable Maximum Storm.—If the probable maximum storm is defined as the largest accumulated precipitation that can reasonably be expected to occur over the basin in a 72-hr period, it is necessary to combine the extreme meteorological parameter values in a reasonable and consistent manner. (The Knox report to the Department of Water Resources uses the term "maximum possible," whereas the term "probable maximum" is also widely used. It is considered that the two terms are synonymous.) This definitely does not imply that all parameters are maximized simultaneously. By combining the parameters in a reasonable manner, an estimated maximum possible 72-hr precipitation total of 19.3 in. for the Feather River Basin was obtained. This total was based upon using assumed rain drops with terminal fall velocity of 8 m per sec. Total water on the windward slope is 34.1 in. and 15.8 in. on the leeward portion of the basin. By assuming a terminal fall velocity of 8 m per sec the amount of water falling on the critical frontal slope of the basin is maximized.

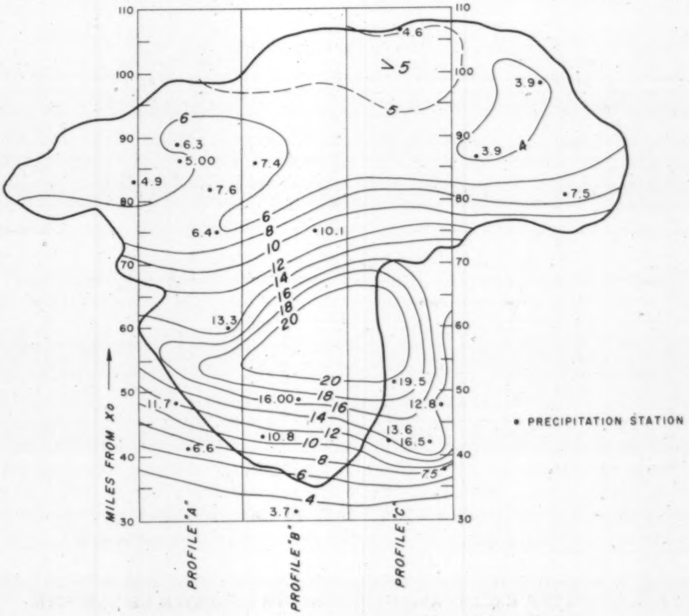
110
100
90
80
70
60
50
40
30
MILES FROM X0
PROFILE "A"
PROFILE "B"
PROFILE "C"
4.6
>5
5
3.9
6
6.3
5.00
7.4
4.9
7.6
3.9
7.5
6.4
10.1
6
8
10
12
14
16
18
20
13.3
20
19.5
11.7
16.00
18
16
12.8
14
13.6
10.8
12
10
16.5
6.6
8
7.5
6
4
3.7
• PRECIPITATION STATION

FIG. 8.—OBSERVED 72-HR ISOHYETAL MAP FOR DECEMBER 1955 STORM, FEATHER RIVER BASIN

Runoff from Probable Maximum Storm.—In order to compare the resulting runoff from the Department's studies with the Corps of Engineers' studies, all unit hydrographs, snowmelt contribution, and loss rates were used as developed in the Corps' hydrology study. However, the area and time distribution of the precipitation as determined by the Knox study was utilized. The resulting hydrograph for Oroville Dam Site has a peak flow of 615,000 cfs and a five-day volume of 2,932,000 acre-ft. The hydrograph and water excess (rain plus snowmelt minus losses) for the storm period are depicted on Fig. 9.

Probable Maximum Runoff Based on U.S. Weather Bureau Storm Studies.—Based on maximum probable storm studies by the Hydrometeorological Section of the U.S. Weather Bureau published in 1943 and revised in 1957, the Department of Water Resources made a preliminary determination in 1957 of the resulting flood runoff.[5] Using the 72-hr basin mean precipitation for the Feather River Basin of 21.1 in. as developed by the United States Weather Bureau Dept. of Commerce (USWB), and assuming conservative values of basin loss rates, snowmelt contribution, and so on, the computed runoff hydrograph resulted in a peak flow of 775,000 cfs and a five-day volume of 3,400,000 acre-ft.

The Sacramento District Corps of Engineers made a detailed analysis of the hydrology of the Feather River Basin in 1958. This analysis, also based on the precipitation data supplied by the USWB, resulted in a probable maximum flood

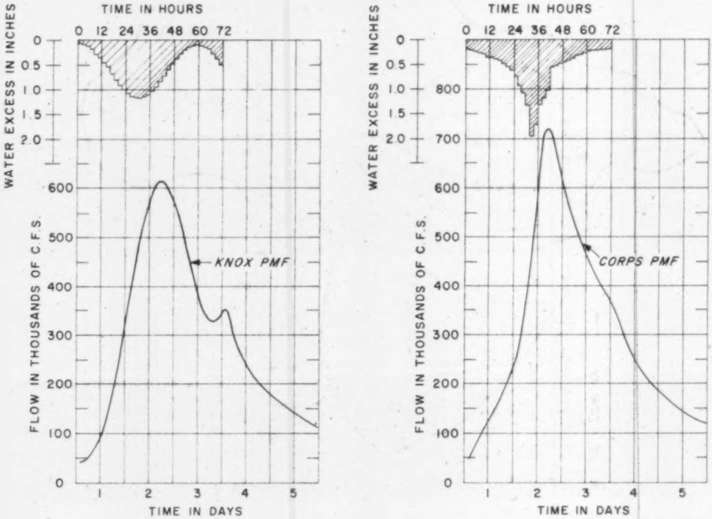

FIG. 9.—WATER EXCESS AND HYDROGRAPHS AT OROVILLE DAM SITE FOR PROBABLE MAXIMUM STORMS

hydrograph (Fig. 9) for the Oroville Dam Site with a peak flow of 720,000 cfs and total volume of 3,063,000 acre-ft. This design flood has been adopted by the Department of Water Resources and has been the basis for hydraulic considerations in the design of the spillway and the establishment of freeboard requirements for Oroville Dam.

Standard Project Flood.—The Sacramento District Corps of Engineers has developed "standard project floods" for the Feather River Basin and the Yuba and Bear Rivers. The following is a definition of standard project rain flood:

> "The standard project rain flood represents the runoff from a standard project rainstorm occurring on ground conditions that are reason-

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[5] "Probable Maximum Flood at Oroville Dam Site," Office Report, State of California, Dept. of Water Resources, Sacramento, Calif., September, 1957.

able likely to exist during storms of such magnitude. The standard project rainstorm for a particular drainage area reflects the most severe flood-producing rainfall depth-area-duration relationship and isohyetal pattern of any storm that is considered reasonably characteristic of the hydrologic region in which the drainage basin is located, consideration being given to the runoff characteristics and effect of water regulation structures in the basin."

The computed peak flows and five-day volumes for the standard project floods for the three basins are shown in Table 1.

TABLE 1.—STANDARD PROJECT FLOOD DATA

Basin (1)	Peak Flow, in cfs (2)	Volume, in acre-feet (3)
Feather River at Oroville	440,000	1,944,000
Yuba River at Mouth	280,000	1,030,000
Bear River at Mouth	91,000	284,000

The Corps of Engineers and the State of California have agreed that the levee system now constructed on the flood plain together with upstream storage should be coordinated so as to provide protection against floods of standard project magnitude.

OROVILLE DAM SPILLWAY

The Oroville Dam Spillway and flood control outlet structure will be located in a topographic saddle on the right abutment of the dam site. The structure will be 700 ft in width and will consist of 380 ft of radial gate-controlled ogee spillway and seven 26.7 ft wide by 26 ft high, low level, flood control release outlets. The flood control release outlets will be in the center of the spillway structure with the gate-controlled overpour spillways on each side as depicted in Fig. 10. The four radial gates in the spillway sections will be 95 ft by 34 ft. The gates will be automatically raised when the water surface exceeds the gross pool elevation of 900 ft and will be fully opened when the water surface reaches 905 ft. The crest of the overflow spillway will be at elevation 868.0 ft United States Geological Survey (USGS) Datum and the flood control outlets will be seated at El. 821.5 ft, USGS Datum. The spillway chute will be concrete lined for a distance of approximately 400 ft downstream from the gate structure. From that point to the end of the spillway chute at the river, approximately 4,000 ft, the spillway channel will be excavated to bed rock and will be unlined. There is a fall of approximately 600 ft in this distance. Rock protection dikes will be required in part of the spillway chute.

Model studies of the spillway were conducted in 1955 by the Department of Water Resources to check design features and hydraulic characteristics of the spillway structure. Fig. 11 is a photograph of the spillway model. Based on these studies, the seven flood control release outlets will pass 249,000 cfs, and the spillway will pass 260,000 cfs, a total of 509,000 cfs, when the reservoir water surface elevation is at 900 ft. With water surface elevation at 909 ft, the

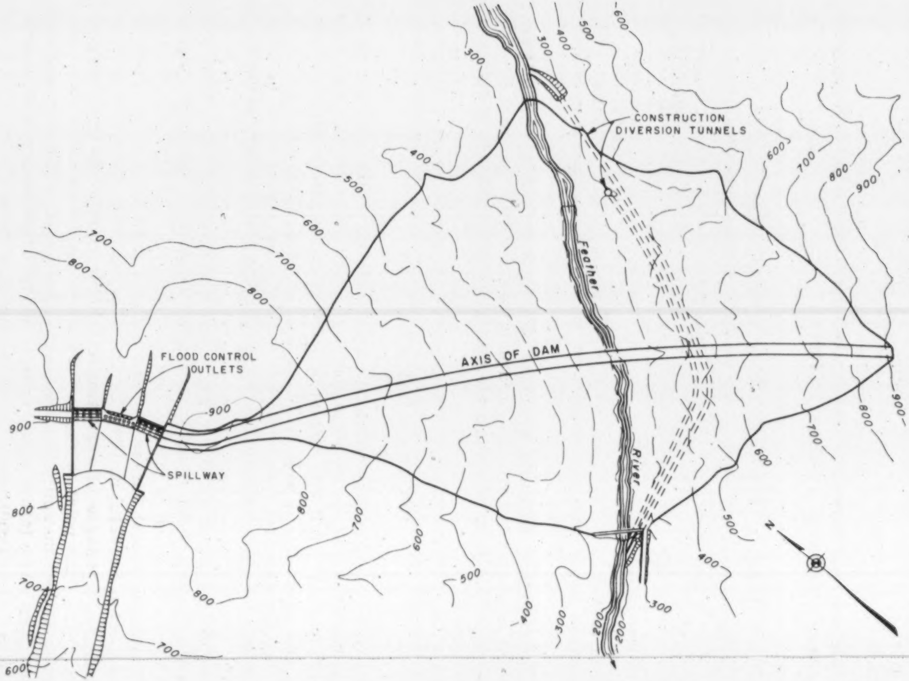
CONSTRUCTION DIVERSION TUNNELS
Feather
River
AXIS OF DAM
FLOOD CONTROL OUTLETS
SPILLWAY
N
200
300
400
500
600
700
800
900

FIG. 10.—PROPOSED OROVILLE DAM AND SPILLWAY

combined discharge will be 650,000 cfs. Assuming that the maximum probable flood occurs with the reservoir water surface at gross pool elevation of 900 ft, the water surface would rise to an elevation of 909 ft. Crest of the dam will be 915 ft. Additional model studies of the spillway design and performance are scheduled by the Department of Water Resources for 1960-61.

FLOW DIVERSION DURING CONSTRUCTION

River diversion during construction will be accomplished by two 38-ft diameter tunnels, located in the left abutment. Floods will be permitted to pass over the dam embankment at the end of the first year of embankment construction if the diversion tunnels are overtaxed. The construction schedule requires that the embankment be placed above elevation 563 ft at the end of the second year of embankment construction in order that the standard project flood could be passed by the diversion tunnels without over-topping the embankment.

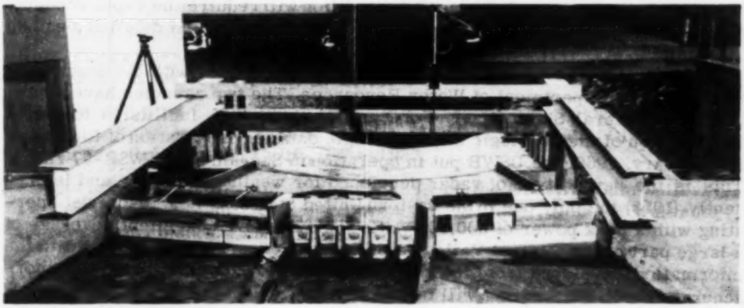

FIG. 11.—MODEL OF OROVILLE DAM SPILLWAY

FLOOD CONTROL OPERATION CRITERIA

To meet the flood control objectives for the Feather River system, flood control releases from Oroville Reservoir must be limited to a maximum release of 150,000 cfs with the occurrence of a flood comparable to the standard project flood. To meet this requirement, the Corps of Engineers has developed a preliminary flood control reservation diagram that requires that a minimum flood control reservation of 375,000 acre-ft must be available between October 1 and April 1 of each year. Based on antecedent rainfall in the basin, a maximum reservation of 750,000 acre-ft may be required between October 15 and April 1. The exact amount of storage between these limits is based on the preceding 60-day basin-mean precipitation. The flood control reservation space can be filled between May 5 and June 15 each year. The exact date of permissible filling is also based on an antecedent rainfall parameter.

It is estimated that with the flood control storage reservation of 750,000 acre-ft, the average annual flood control benefit accruing to the Oroville Dam Project will be $3,640,000. Preliminary estimates of the federal government participation in construction costs of Oroville Dam and Reservoir, based on this flood control benefit, will be approximately 26% of the construction costs, exclusive of the cost of power facilities. This contribution, based on present (1959) cost estimates, will be approximately $77,000,000.

OPERATION OF OROVILLE DAM

The physical operation of Oroville Dam and Reservoir during flood periods will require special attention because of the very rapid runoff from the frontal slope of the Feather River Basin. Under existing conditions, the peak flow occurs at the dam site only 4 hr to 6 hr after the heaviest burst of precipitation. With the dam completed, this time will be shortened because of the effect of the extensive reservoir area. This condition will require the rapid accumulation of precipitation data from the basin and accurate and detailed weather forecasts.

The USWB in Sacramento is located in adjoining offices with the Operations Branch of the Department of Water Resources. The two agencies have established a Federal-State River Forecast Center which facilitates the rapid assimilation of meteorologic and hydrologic data and preparation of forecasts. In February, 1960, the USWB put in operation in Sacramento a WSR-57 radar. This is the newest type of radar developed for weather detection and is presently (1959) the only one in the western United States. It is a 10-cm set operating with a peak power of 500,000 watts. This set gives excellent coverage of a large part of the Central Valley watershed. As experience is gained with the information developed from the radar installation, it is anticipated that more accurate runoff forecasting will be possible.

In addition to an extensive radio reporting stream gage network operated by the Department of Water Resources in the Central Valley, an experimental radio reporting rain gage was installed at Brush Creek in 1959. This gage has proved to be very reliable. (The precipitation data recorded on Fig. 3 was obtained from this radio reporting rain gage.)

It is anticipated that the combination of the radar and radio reporting river and rain gages will provide the hydrologists and meteorologists with the information to operate efficiently Oroville Dam and Reservoir and to integrate its operation with the operation of the reservoir and levee systems in the Central Valley.

ACKNOWLEDGMENTS

The contribution to the field of hydrometeorology by Joseph B. Knox and his work with the Department of Water Resources are gratefully acknowledged. The cooperation of the Sacramento District, Corps of Engineers, is also appreciated. The figures presented herein were drawn by Mr. Kay Shibata, Delineator, Department of Water Resources, State of California.

APPENDIX.—ADDITIONAL REFERENCES

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3. "Northeastern Counties Investigation," State of Calif., Dept. of Water Resources, Bulletin No. 58, Sacramento, Calif., December, 1957.
4. "Maximum Possible Precipitation, Sacramento River Basin," U. S. Weather Bur., Hydrometeorological Report No. 3, Washington, D. C., May 15, 1942 (revised by letter report, June 6, 1957).
5. "Design and Cost Estimates of Oroville Reservoir, Dam, Power Plant and Afterbays," Office Report, State of Calif., Dept. of Pub. Wks., Div. of Water Resources, Sacramento, Calif., June, 1955.
6. "Report on the Hydraulic Model Studies of the Oroville Dam Spillway, Feather River Project," Office Report, State of Calif., Dept. of Water Resources, Sacramento, Calif., March, 1955.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3213

LABORATORY INVESTIGATION OF RUBBLE-MOUND BREAKWATERS

Robert Y. Hudson,[1] F. ASCE

With Discussion by Messrs. José Reis de Carvalho and Daniel Vera-Cruz; William H. Booth, Jr.; Francis B. Slichter; Leland B. Jones; Omar J. Lillevang; Thorndike Saville, Jr.; and Robert Y. Hudson.

SYNOPSIS

This paper reports on a laboratory investigation conducted at the United States Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, to determine criteria for the design and construction of rubble-mound breakwaters. Small-scale breakwater sections are hand-constructed in a concrete wave flume 119 ft long, 5 ft wide, 4 ft deep, and subjected to mechanically generated waves to determine the stability of the armor units.

A general stability equation has been derived and is being used to guide the experimental program and correlate the test data. From the test data obtained important unknown functions in the general stability equation have been determined for selected breakwater and test-wave conditions, and a new breakwater stability formula has been obtained.

In conjunction with the stability tests, wave run-up data are obtained for each breakwater section and wave condition tested. Also, measurements are obtained that enable the thickness and porosity of cover layers composed of different types of armor units to be determined.

The new stability formula and the experimental data obtained so far have provided essential information for an improved method of designing rubble-mound breakwaters with protective cover layers composed of quarry-stone and tetrapod armor units. Tests in progress (1959) to obtain experimental data for other special shapes of cast-concrete armor units (cubes, tetra-

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hedrons, and tribars) should increase considerably the accuracy of rubble-mound breakwater design.

INTRODUCTION

Small-scale tests of rubble-mound breakwaters have been in progress at the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, almost continuously since 1942. During the period from 1942 to 1950, various phases of rubble-mound breakwater construction were investigated for the Bureau of Yards and Docks, Department of the Navy. The most important findings of that investigation concerned the accuracy of Iribarren's formula (1),(2),(3).[2] It was concluded (4) that the Iribarren formula can be used for the design of rubble-mound breakwaters only if experimental coefficients, of the kind developed during the investigation conducted for the Bureau of Yards and Docks, are available for the complete range of variables encountered in the design of full-scale structures.

In 1951, a comprehensive investigation of rubble-mound breakwaters (5) was begun at the Waterways Experiment Station for the Office, Chief of Engineers, U. S. Army. This investigation (in progress in 1959) is similar to the study conducted for the Bureau of Yards and Docks except that it is larger in scope; it includes the necessary range of important variables that affect the stability of rubble-mound breakwaters.

To insure optimum designs for breakwaters, design engineers should have accurate information concerning the required weight for the individual armor units in the protective cover layer, along the length of the structure, as a function of; (a) shape of unit, (b) specific weight of unit, (c) specific weight of water in which the structure will be situated, (d) beach slope seaward of the breakwater, (e) dimensions of waves at the location of the proposed structure, (f) seaside slope of breakwater, (g) porosity of protective cover layer, (h) thickness of cover layer, and (i) porosity and thickness of underlayers on which the armor units are to be placed. In addition, design engineers should be able to determine quantitatively; (a) the height of breakwater above still-water level necessary to prevent excessive overtopping by wave run-up, (b) the depths below still-water level to which the cover layer should extend, (c) the amount of damage that will be inflicted on a breakwater section not designed for overtopping when waves higher than the selected design wave occur, and (d) the best design of back slopes for preventing failure when overtopping of the breakwater is permitted. Information should also be available for designing the seaward end, or head, of the breakwater.

The test program under discussion includes tests to provide the design data and quantitative information that has been outlined. However, tests described in this paper are concerned, for the most part, with the types of rubble-mound breakwaters in which that part of the breakwater section subjected to the most intense wave action is composed of a pile of quarrystone armor units placed pell-mell, and those in which the protective cover layers are composed of two layers of cast-concrete armor units placed pell-mell over one or two quarry-stone underlayers.

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[2] Numerals in parentheses—thus, (1)—refer to corresponding items in the Bibliography—see Appendix I.

After the comprehensive investigation was begun, it was found that the Iribarren formula has limitations that render it unsatisfactory for use in correlating stability data from tests of small-scale rubble-mound breakwaters. Thus, it was necessary to reanalyze the phenomenon that results when waves attack a rubble-mound breakwater in order to develop a more general stability equation.

This paper describes the apparatus and testing techniques used in the laboratory investigation, explains why it was considered necessary to abandon the use of Iribarren's formula in correlating test data, and presents the derivation of a more general stability equation that, with the experimental data obtained to date, was used to develop a simple formula for the weight of armor units necessary to insure the stability of rubble-mound breakwaters. Information concerning wave run-up, and the thickness and porosity of cover layer materials is also presented.

For this paper, a rubble-mound breakwater is considered to be one constructed with a core of quarry-run stones, sand, slag, or other suitable materials, protected from wave action by one or more stone underlayers and a cover layer of relatively large, selected quarry stones or specially-shaped concrete armor units.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically, for convenience of reference, in Appendix II.

DISCUSSION OF IRIBARREN'S FORMULA

Iribarren's original formula for the weight of armor units in rubble-mound breakwaters, in its general form, revised (6) to make it dimensionally homogeneous, and retaining the coefficient of friction as a variable, reduces to

$$W_r = \frac{K' \gamma_r \mu^3 H^3}{(\mu \cos\alpha - \sin\alpha)^3 (S_r - 1)^3} \qquad \ldots\ldots\ldots\ldots (1)$$

in which W_r is the weight of individual armor units, γ_r is the specific weight of the armor units, S_r is the specific gravity of the armor units relative to the water in which the breakwater is situated $(S_r = \gamma_r/\gamma_w)$, μ is the effective coefficient of friction between armor units, H is the height of wave attacking the breakwater, α is the angle, measured from the horizontal, of the exposed breakwater slope, and K′ is an experimentally determined coefficient. The accuracy of this formula was discussed by Hudson and Jackson (4), and Hudson (6) in 1953. At that time it was concluded that the Iribarren formula could be used to correlate the test data, and that it could be made sufficiently accurate for use in designing full-scale rubble-mound breakwaters, if sufficient test data were available to evaluate the experimental coefficient (K′).

After the comprehensive testing program was begun, and shortly after the conclusions concerning the adequacy of Iribarren's formula were published, preparations were initiated for tests to determine the stability of armor units as a function of armor-unit shape. These included a study to establish the values of the friction coefficient (μ) that should be used for the various shapes of armor units in the experimental determination of K′ in Iribarren's formula. The first armor units of special shape for which friction coefficients were measured were cubes and tetrapods. Tetrapod is the name of a patented armor unit of special shape that was developed at the Laboratoire Dauphinois d'Hydraulique Ets. Neyrpic, Grenoble, France (7). The tests showed that the friction coefficient in Iribarren's formula, as measured by the tangent of the angle of repose (ϕ), varied appreciably with the shape of armor unit and the method of placing these units in the cover layer. These results led to the realization that the experimental coefficient (K') in Iribarren's formula could not be determined accurately from small-scale breakwater stability tests unless accurate comparative values of the friction coefficient could be obtained for the different shapes of armor units. This realization was made more acute by the fact that Iribarren's force diagram, from which his basic stability equation was derived, is predicated on the assumption that the friction between armor units, specifically that component of the friction force parallel to the breakwater slope, is the primary force that resists the forces of wave action and determines the stability of the armor units.

Results of coefficient-of-friction determinations for three sizes of quarry stones, and for concrete cubes and tetrapods are shown in Table 1. Fig. 1 shows the shapes of these armor units. About seventy repeat tests of the 0.30-lb, quarry-stone armor units were conducted to determine the range of μ for units of this type. It was found that μ varied from a low of 0.78 to a high of 1.28, with an average value of 0.98. Thus, μ varies not only with armor-unit shape and method of placing, but it also varies considerably from test to test for the same armor unit. The curves of Fig. 2 were prepared using the modified Iribarren formula (Eq. 1), and show the effects of variations in the measured value of μ on the computed values of K'. Because W_r is directly proportional to K', variations in μ have the same effect on computed values of W_r as they do on K'. It can be seen that for steep breakwater slopes, small variations in the measured value of μ cause large variations in the computed values of K' and W_r. This becomes more significant when it is recalled that the use of concrete armor units of special shape is more apt to be economically feasible only for the steeper breakwater slopes.

TABLE 1.—FRICTION COEFFICIENTS OF ARMOR UNITS

No.	Method of Measurement	Quarry Stone			Concrete Cubes	Concrete Tetrapods
	(1)	W_r = 0.10 lb (2)	W_r = 0.30 lb (3)	W_r = 0.62 lb (4)	W_r = 0.80 lb (5)	W_r = 0.21 lb (6)
①	Dumped in water	1.02	0.98	1.13	1.20	1.10
②	Dumped in air	0.79	0.90	0.87	1.34	---
③	Stacked in water	1.09	1.19	1.26	1.36	1.78
④	Stacked in air	0.97	1.12	1.22	1.75	---
	Avg (all methods)	0.97	1.05	1.12	1.41	---
	Avg (① and ③)	1.06	1.09	1.20	1.28	1.44

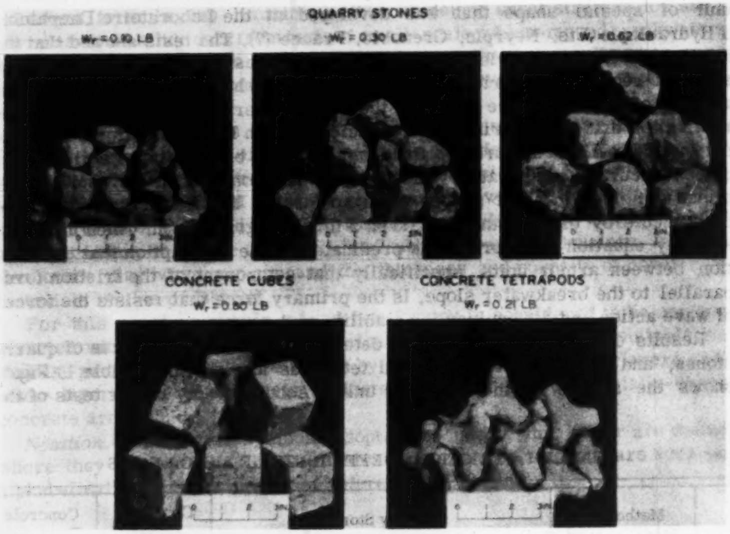
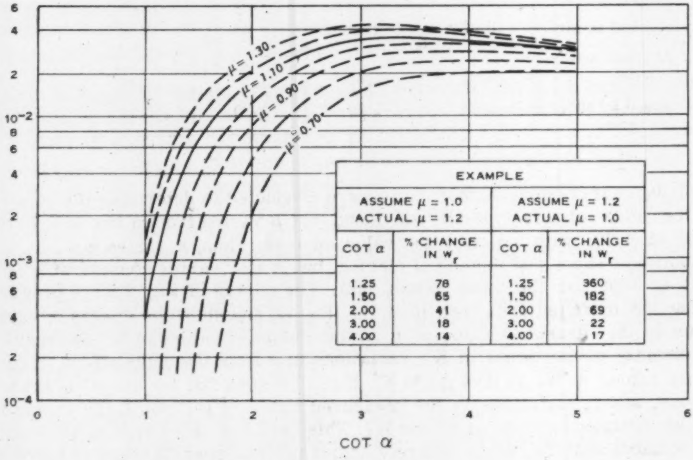

QUARRY STONES
W_r = 0.10 LB
W_r = 0.30 LB
W_r = 0.62 LB
0 1 2 3IN
CONCRETE CUBES
W_r = 0.80 LB
CONCRETE TETRAPODS
W_r = 0.21 LB

FIG. 1.—TYPES OF ARMOR UNITS FOR WHICH FRICTION COEFFICIENTS WERE DETERMINED



EXAMPLE			
ASSUME μ = 1.0 ACTUAL μ = 1.2		ASSUME μ = 1.2 ACTUAL μ = 1.0	
COT α	% CHANGE IN W_r	COT α	% CHANGE IN W_r
1.25	78	1.25	360
1.50	65	1.50	182
2.00	41	2.00	69
3.00	18	3.00	22
4.00	14	4.00	17

FIG. 2.—VARIATIONS OF K' WITH μ IN THE MODIFIED IRIBARREN FORMULA

Based on the results of the tests to determine friction coefficients, correlation of test data by the use of Iribarren's formula was abandoned, and a new stability equation, similar to the Iribarren formula but capable of more general application, was derived.

ANALYTICAL BASES OF STABILITY EQUATION

When short-period wind waves impinge on a pervious rubble-mound breakwater, the resulting interplay of forces developed by the wave-induced water motion and the resisting action of the armor units in the cover layer is extremely complex, and attempts to describe the phenomenon quantitatively by rigorous theoretical analyses have not, as yet, been successful. Waves at a breakwater may break completely, projecting a jet of water approximately perpendicular to the slope, break partially with a poorly defined jet, or establish an oscillatory motion of the water particles along the breakwater slope similar to the motion of a clapotis at a vertical wall. Characteristics of the motion of water particles when short-period wind waves encounter a rubble-mound breakwater are determined by the wave steepness (H/λ), the relative depth (d/λ), the relative height (H/d), the depth of water at the toe of the breakwater slope (d), the angle of the beach slope seaward of the breakwater (σ), angle of seaside slope of the breakwater with the horizontal (α), the angle of obliquity of the attacking waves (β), and the shape, thickness, and porosity of the cover layer and underlayer materials $(\Delta$, r, and P, respectively).

The ability of an armor unit in the cover layer to resist the forces caused by wave action is determined by the buoyant weight of the armor unit (W_r'), the position of the unit relative to the still-water level (z), the angle of seaside slope (α), the height of breakwater crown above still-water level (h), the width of breakwater crown (m), the shape of unit (Δ), porosity of the armor units in place (P), thickness of the cover layer (r), the porosities and thicknesses of the underlayers, and the method of placing the breakwater material, especially the armor units in the cover layer (dumped pell-mell, placed in some orderly manner to obtain wedging action, or stacked without wedging action).

Short-period wind waves incident on a rubble-mound breakwater develop dynamic forces that tend to lift and roll the armor units from the breakwater slope. These forces consist of a drag force

$$F_d = \frac{1}{2} C_d \, k_a \, l^2 \, \frac{\gamma_w}{g} \, V^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

and an inertia force

$$F_m = C_m \, k_v \, l^3 \, \frac{\gamma_w}{g} \, \frac{\partial V}{\partial t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which C_d is a drag coefficient, C_m is a virtual-mass coefficient, l is a characteristic linear dimension of the unit such that the projected area of the unit perpendicular to the velocity is $k_a \, l^2$, and the volume of the unit is $k_v \, l^3$, γ_w is the specific weight of the water in which the breakwater is to be situated, g is acceleration due to gravity, and V is the velocity of the water flowing around or impinging on the armor units in the cover layer. Because of the difficulties inherent in an attempt to evaluate the separate sets of coefficients,

$C_d\ k_a$ and $C_m\ k_v$, that would involve either direct measurement or a derived expression of the acceleration $(\partial V/\partial t)$ in terms of the wave characteristics, and in order to simplify the force equation used to correlate test data, the effects of acceleration are combined with the drag force. The resulting equation is

$$F_q = C_q\ l^2\ \frac{\gamma_w}{g}\ V^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which C_q, the total coefficient, is a function of the terms $\frac{l}{V^2}\ \frac{\partial V}{\partial t}$, $C_d\ k_a$, and $C_m\ k_v$.

The velocity of the water jet resulting from a breaking wave (V_b) is equal to the particle velocity at the wave crest that, at the instant of breaking, is equal to the celerity of the wave form. Thus, for shallow-water waves, as $d/\lambda \longrightarrow 0$,

$$V_b^2 = g\ d_b \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Also at breaking, $H_b = k\ d_b$, in which $k = f(H/\lambda)$. Therefore, by substitution,

$$V_b^2 = \frac{g}{k}\ H_b \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Substituting this value of velocity in Eq. 4, the expression for the force exerted on an armor unit by a breaking wave, in terms of wave height, is

$$F_q = C_q\ l^2\ \frac{\gamma_w}{k}\ H_b \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

For breakwaters constructed by dumping or by placing armor units essentially pell-mell, the forces resisting displacement are the buoyant weight of the individual units and the friction between units. Except for isolated instances in which wedging action is involved, friction between armor units can be neglected, and the principal resisting force for pell-mell-constructed cover layers can be assumed to be

$$W_r' = k_v\ l^3 (\gamma_r - \gamma_w) \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which γ_r is the specific weight of the armor units.

For incipient instability of armor units in a rubble-mound breakwater, or fill slope, subjected to breaking waves, $W_r' = F_q$, or

$$k_v\ l^3 (\gamma_r - \gamma_w) = C_q\ l^2\ \frac{\gamma_w}{k}\ H_b \quad \ldots\ldots\ldots\ldots (9)$$

Letting $S_r = \gamma_r/\gamma_w$, and substituting in Eq. 9,

$$k_v\ l (S_r - 1) = \frac{C_q\ H_b}{k}$$

or

$$\frac{H_b}{l (S_r - 1)} = \frac{k\ (k_v)}{C_q} \quad \ldots\ldots\ldots\ldots\ldots (10)$$

The weight of an armor unit in air is $W_r = k_v\ l^3\ \gamma_r$, or

$$l = \left(\frac{W_r}{k_v\ \gamma_r}\right)^{1/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Substituting this value of l in Eq. 10,

$$\frac{\gamma_r^{1/3}\, H_b}{\left(S_r - 1\right) W_r^{1/3}} = \frac{k\left(k_v\right)^{2/3}}{C_q} \quad \ldots\ldots\ldots\ldots\ldots (12)$$

in which

$$\frac{k\left(k_v\right)^{2/3}}{C_q} = f\left(C_d,\ k_a,\ C_m,\ k_v,\ \frac{1}{V^2}\,\frac{\partial V}{\partial t},\ d/\lambda,\ H/\lambda\right)$$

The forces that tend to displace armor units from breakwater slopes when the waves do not break, or break only partially, are not the same as those forces that result from breaking waves, nor do they act in the same directions. However, the order of magnitude of the nonbreaking wave forces, and the effects of these forces on the stability of rubble-mound breakwaters, should be approximately the same as those caused by breaking waves. It is believed, therefore, that Eq. 12 adequately represents, at least in the first approximation, the major forces of both breaking and nonbreaking waves. Thus, for both types of short-period wave motions on rubble-mound breakwaters, and introducing those variables that were not included in the derivation of Eq. 12, the most general equation used in this investigation to guide the testing program and correlate test data is

$$\frac{\gamma_r^{1/3}\, H}{\left(S_r - 1\right) W_r^{1/3}} = f\left(\begin{array}{l} \alpha,\ C_d,\ C_m,\ k_a,\ k_v,\ \dfrac{1}{V^2}\,\dfrac{\partial V}{\partial t},\ H/\lambda,\ d/\lambda, \\ H/d,\ d,\ \sigma,\ P,\ r,\ h,\ m,\ z,\ \beta, \text{ and} \\ \text{the method of placing armor units} \end{array}\right) \quad \ldots (13)$$

In Eq. 13, C_d and C_m are functions of Δ and the Reynolds number $(\mathbf{R})$, and k_a and k_v are functions of Δ. The term $\frac{1}{V^2}\frac{\partial V}{\partial t}$, that is a form of Iversen's modulus for accelerated motion (8), is omitted from the list of variables tested in this investigation because of the difficulty of obtaining accurate velocity-time histories of the flow around individual armor units.

In the first phase of this testing program, the upper portion of the small-scale breakwaters was constructed of rocks simulating quarry stones, all pieces of which were of nearly the same weight, specific weight, and shape. In addition, the crown width of the breakwater test sections was standardized at three times the average diameter of the armor units; the angle of obliquity of the test waves was 0°, and the cover layer was extended to a depth below still-water level sufficient to insure that the stability of the structure would not be influenced by the stones used in the lower portion of the test section. For those tests in which the no-damage criterion was used in the selection of design-wave heights, the crown heights above still-water level were sufficient to prevent overtopping by the test waves. For those tests in which the wave heights used were greater than the previously selected design-wave heights, the crown heights above still-water level, and the depths to which the cover layers extended below still-water level, were equal to the previously selected design-wave heights. For all tests, the water depth between the wave generator and the breakwater was constant, and was sufficient to prevent the ratio H/d from influencing the action of waves on the structure. For the tests conducted, the variation in Reynolds number was comparatively small. Tests in a larger wave flume at the laboratory of the Beach Erosion Board, Washington, D. C., are being conducted to determine the effects of this variable on the stability of armor units in rubble-mound breakwaters.

When damage is allowed to occur to the breakwater (by use of wave heights greater than the design-wave height), the geometry of the structure, the motion of the water particles, and the resulting forces on the breakwater differ from those resulting from tests in which the no-damage criterion is used. Thus, a damage parameter, D, defined as the percentage of armor units displaced from the cover layer by wave action, is included as a prime variable.

For the breakwater sections investigated in the first phase of the testing program, in which the armor units were rocks simulating rounded and smooth quarry stones placed pell-mell

$$\frac{\gamma_r^{1/3} H}{(S_r - 1) W_r^{1/3}} = f(\alpha, H/\lambda, d/\lambda, \text{and } D) \quad \ldots\ldots\ldots\ldots (14)$$

In the second phase of the testing program the armor units used were patterned after the tetrapod, and the rubble mound was protected by two or more layers of armor units placed over one or two quarry-stone underlayers. For these tests

$$\frac{\gamma_r^{1/3} H}{(S_r - 1) W_r^{1/3}} = f(\alpha, H/\lambda, d/\lambda, r) \quad \ldots\ldots\ldots\ldots (15)$$

The dimensionless parameter on the left side of Eqs. 13 through 15 is designated the stability number (N_s) for rubble-mound breakwaters.

EXPERIMENTAL EQUIPMENT AND PROCEDURE

Test Apparatus.—The breakwater stability tests are conducted in a concrete flume 5 ft wide, 4 ft deep, and 119 ft long, equipped with a plunger-type wave generator. Wave heights are measured with a parallel-rod-type wave gage, and recorded on a direct-writing oscillograph. The wave-height measuring apparatus consists of the wave gage (two 1/8-in. stainless steel, parallel rods 1.2 ft long, spaced 2 in. apart), a balancing circuit, a Brush universal analyzer, and a magnetic oscillograph.

Cross-section measurements of the small-scale breakwaters are obtained with a sounding rod equipped with a circular spirit level for plumbing, a scale graduated in thousandths of a foot, and a ball-and-socket foot that facilitates adjustment to the irregular surface of the breakwaters. The foot is circular, and for each test the diameter of the foot is equal to one-half the average diameter of the armor units.

Types of Tests Conducted.—Two primary types of stability tests are being conducted in this investigation. First, design-wave heights are determined for breakwater sections of sufficient height to prevent overtopping by the test waves. Design-wave height is defined as the maximum wave height, measured at the location of a proposed breakwater before it is constructed, that will not damage the cover layer. The removal of as much as 1% of the total number of armor units in the cover layer is considered to be "no damage."

The second type of tests being conducted is concerned with determination of safety factors for breakwater sections designed on the basis of the criteria established from results of the no-damage and no-overtopping tests. For the safety-factor tests, breakwater sections are constructed in the wave flume in accordance with the results of the no-damage and no-overtopping tests, and the amount of damage, as determined by the percentage of armor units removed from the cover layer, is obtained as a function of wave height. Wave heights greater than the previously selected design-wave height for the no-damage and no-overtopping criteria are used in these tests.

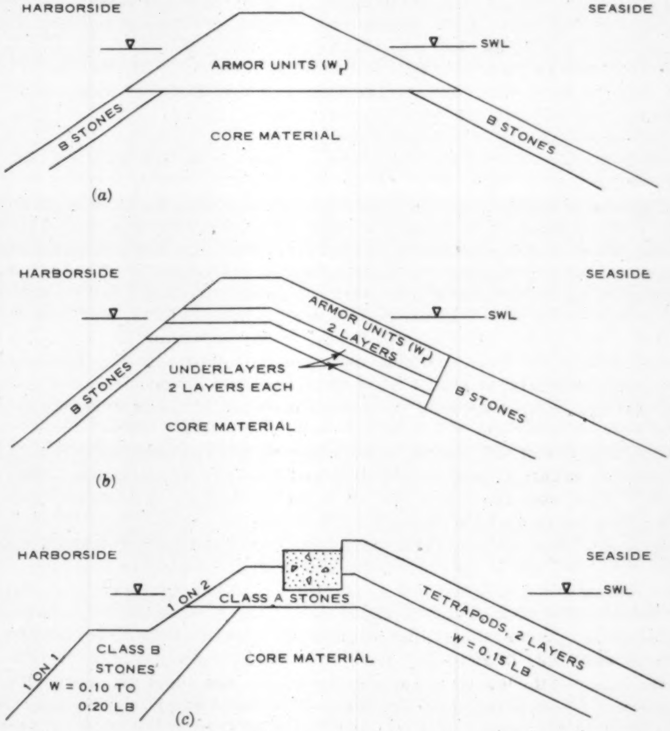

HARBORSIDE
SEASIDE
SWL
ARMOR UNITS (W_r)
B STONES
CORE MATERIAL
B STONES
(a)
HARBORSIDE
SEASIDE
SWL
ARMOR UNITS (W_r) 2 LAYERS
UNDERLAYERS 2 LAYERS EACH
B STONES
B STONES
CORE MATERIAL
(b)
HARBORSIDE
SEASIDE
SWL
1 ON 2
CLASS A STONES
TETRAPODS, 2 LAYERS
W = 0.15 LB
1 ON 1
CLASS B STONES
W = 0.10 TO 0.20 LB
CORE MATERIAL
(c)

FIG. 3.—ELEMENTS OF BREAKWATER SECTIONS TESTED

In addition to the two previously mentioned types of tests, special tests are conducted from time to time to determine optimum designs for specific breakwaters. In these tests, design-wave heights may be determined for conditions other than no-damage and no-overtopping.

Breakwater Sections Tested.—Rubble-mound breakwaters of the types shown schematically in Figs. 3(a) and 3(b) have been used in most of the stability tests. In the no-damage and no-overtopping tests, the crown heights were sufficient to prevent overtopping, and the cover layer was extended a sufficient distance below still-water level to prevent damage to the class B stones used below the armor units. The distance below still-water level to which the armor units extended, as well as the height of the breakwater crown above still-water level, was equal to or greater than the wave heights used to test the breakwater sections. In the safety-factor tests of quarry-stone armor units, the crown heights above still-water level, and the maximum distances below still-water level to which the armor units in the cover layers extended, were numerically equal to the design-wave heights previously selected in the corresponding no-damage and no-overtopping tests.

In a few tests to determine the stability of the Crescent City Harbor breakwater (9), the type of breakwater section shown in Fig. 3(c) was used. This breakwater section was designed for overtopping.

Types of Breakwater Materials Used.—

Quarry-stone armor units and class B stones.—In each stability test the quarry-stone armor units were as nearly the same weight, specific weight, and shape as possible. Both the armor stones and class B stones were sized from crushed basalt. The weights of class B stones were approximately the same as those of the armor stones; however, the class B stones were sized by means of sieves, whereas each armor stone was sized and shaped by hand and weighed on a torsion balance having a sensitivity of 0.1 g. Two sizes of armor stones were used to insure that the design-wave heights, and the heights of waves used in the safety-factor tests, would be within the range of wave dimensions that the wave machine can generate. Approximately 2800 pieces of the larger-size armor stones were used. Based on a representative sample of 175 pieces, the average weight and specific weight of the larger-size armor stones were 0.30 lb and 176.0 lb per cu ft, respectively. Based on a representative samples of 475 pieces, the average weight and specific weight of the smaller-size armor stones were 0.10 lb and 174.7 pcf, respectively. The core material, that was the same for all tests conducted, consisted of crushed basalt with a mean particle diameter of 1/8 in.

Tetrapod armor units.—Tests have been conducted using tetrapod-shaped armor units molded of both concrete and leadite. Leadite is the trade name for a caulking compound that has a specific weight nearly the same as that of the concrete used to mold the tetrapod armor units. Based on representative samples of 125 pieces, the average weight and specific weight of the concrete tetrapods were 0.21 lb and 142.3 pcf, respectively, and the average weight and specific weight of the leadite tetrapods were 0.22 lb and 140.4 pcf, respectively.

Method of Constructing Test Sections.—The breakwater test sections were constructed in the wave flume on a sand base 85 ft from the wave generator. The core material and class B stones from the base of the test section to the crown of the core were placed with the flume dewatered. The core material was wetted with a hose and then compacted with hand trowels to simulate the natural consolidation effected by wave action during construction of full-scale structures. The class B stones were then placed by shovel and dressed by hand, after which the flume was flooded to the desired still-water level. For the type of breakwater shown in Fig. 3(a), the quarry-stone armor units from the crown of the class B stones and core-material section to the still-water level (swl) were placed by dumping, pell-mell, from a container at the water surface. Above the still-water level the quarry-stone armor units were placed by hand. For the types of breakwater illustrated in Figs. 3(b) and 3(c), the class A and class B stones and the core material were placed in the manner described for placement of the class B stones, and the armor units, both above and below the water surface, were placed by hand. These methods of constructing the breakwater test sections were adopted so as to reproduce, as nearly as possible, the usual methods of constructing full-scale structures.

Selection of Design-Wave Heights.—Design-wave heights for the no-damage $(H_D = 0)$ criterion were determined by subjecting the test sections to waves made successively higher, in approximately 0.02-ft increments, until the maximum wave height was found that would not remove more than 1% of the armor units from the cover layer. Each size wave was allowed to attack the breakwater for a cumulative period of 30 min, after which the test sections were rebuilt prior to attack by the next added-increment wave.

TABLE 2.—RANGES OF WAVE AND BREAKWATER CHARACTERISTICS TESTED

Characteristic (1)	Range of Test Conditions (2)
Wave height (H)	0.28 to 0.69 ft
Water depth (d)	1.26 and 2.00 ft
Wave period (T)	0.88 to 2.65 sec
Wave length (λ)	4.0 to 20.0 ft
Relative depth (d/λ)	0.10 to 0.50
Wave steepness (H/λ)	0.015 to 0.128
Specific weight of:	
Quarry stones (γ_r)	166.0 to 191.6 lb per cu ft
Concrete tetrapods (γ_r)	135.0 to 154.0 lb per cu ft
Leadite tetrapods (γ_r)	134.0 to 142.0 lb per cu ft
Water (γ_w)	62.4 lb per cu ft
Weight of:	
Quarry stones (W_r)	0.09 to 0.31 lb
Concrete tetrapods (W_r)	0.18 to 0.24 lb
Leadite tetrapods (W_r)	0.21 to 0.23 lb
Breakwater slope ($\tan \alpha$)	1 on 1.25 to 1 on 5

Range of Test Conditions.—The tests involved the ranges of wave and breakwater characteristics listed in Table 2.

Test Waves.—During the tests, the wave generator was stopped as soon as reflected waves from the breakwater reached it, and the waves were allowed to decay in order to prevent the test section from being exposed to a multiple, undefined wave system. Accurate determination of the height of test waves was complicated by the presence of waves of abnormal height in the train of waves, caused by the starting and stopping of the generator. Usually there were one or two large waves at the end of each cycle. The larger waves, which occurred approximately 1% of the time that waves attacked the test structure, averaged about 12% higher than the average height of the highest one-third of the waves in the wave trains $(H_{1/3})$. Waves of height $H_{1/3}$ are called the "significant" waves of fully established wave trains in nature. It has been determined (10) that storm-wave trains in nature contain waves about 25% larger than the significant wave 5% of the time, 33% larger 3% of the time, and 58% larger 1% of the time. The importance of these facts with respect to the design of rubble-mound breakwaters is not fully understood at the present time. However, it is believed that the existence of these larger-size waves in natural wave trains must be considered in the selection of design-wave heights and factors of safety.

RESULTS OF STABILITY TESTS

No-Damage Conditions.—Data obtained from stability tests of quarry-stone and tetrapod-shaped armor units for the no-damage criterion are shown in Fig. 4 in the form of a log-log plot, with the stability number as the ordinate, cot α as the abscissa, and the shape of the armor unit as the parameter. These data consist of experimentally determined design-wave heights and corresponding computed stability numbers, as functions of breakwater slope and shape of armor unit. Data concerning quarry-stone armor units were obtained for breakwater sections of the type shown in Fig. 3, and the design-wave heights were determined for the no-damage and no-overtopping criteria. Data concerning tetrapods, using the no-damage and no-overtopping criteria, were obtained for breakwater sections of the type shown in Fig. 3(b). Data

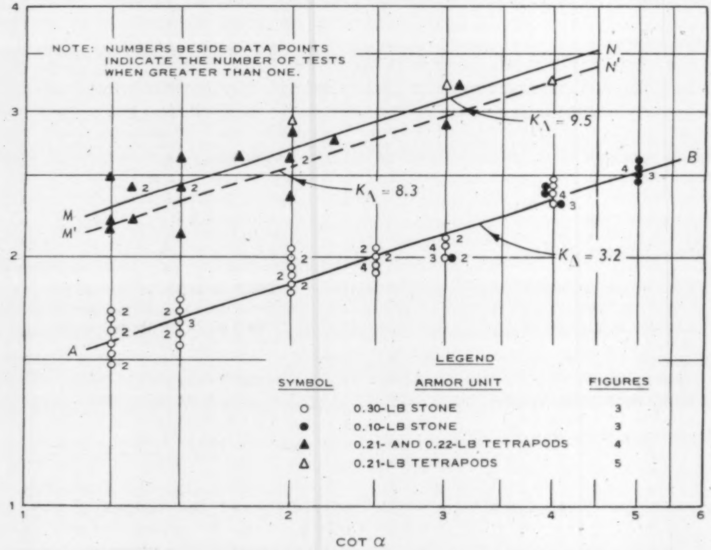


FIG. 4.—STABILITY OF QUARRY-STONE AND TETRAPOD ARMOR UNITS: N_s AS A FUNCTION OF Δ AND α FOR THE NO-DAMAGE AND NO-OVERTOPPING CRITERIA

were also obtained for a breakwater section of the type shown in Fig. 3(c), using the no-damage criterion. The crown of the latter breakwater section was designed for overtopping.

Analysis of the test data indicated that, for the conditions tested, the effects of the variables d/λ and H/λ on the stability of armor units are of second order in importance compared with the effects of breakwater slope and shape of armor unit. A formula for determining the weight of armor units necessary to insure stability of rubble-mound breakwaters of the types tested, and in relatively deep water, can be obtained from the equation of the approximate best-fit lines in Fig. 4. The lines AB and MN were drawn through the data points using a slope of one-third to simplify the derived formula. The equation of a straight line on log-log paper is of the form $y = a\,x^b$, in which a is the y intercept at $x = 1$, and b is the slope of the line. The equation of lines AB and MN, therefore, is

$$N_s = a\,(\cot\,\alpha)^{1/3} \quad \text{. . . . .(16)}$$

or

$$\frac{\gamma_r^{\,1/3}\;H_{D=0}}{W_r^{\,1/3}\left(S_r - 1\right)} = a\;(\cot\,\alpha)^{1/3} \quad \text{. . . . .(17)}$$

from which, if $K_\Delta = a^3$,

$$W_r = \frac{\gamma_r\left(H_{D=0}\right)^3}{K_\Delta\left(S_r - 1\right)^3\,\cot\,\alpha} \quad \text{. . . . .(18)}$$

This is the desired stability formula for quarry-stone and tetrapod-shaped armor units for the no-damage and no-overtopping conditions. The test data indicate that, for pell-mell placing of armor units, the experimentally determined coefficient (K_Δ) varies primarily with shape of the armor units. The values of K_Δ for quarry-stone and tetrapod-shaped armor units, corresponding to the best-fit lines AB and MN of Fig. 4, are 3.2 and 9.5, respectively.

Tests conducted previously showed that, for the type of breakwater tested and for breakwater slopes flatter than 1-on-2, the stability number increases slightly as the number of layers of armor units is increased from 2 to 4. Although an increase in stability number means a decrease in weight of armor unit for the same wave height, the saving in volume of material per armor unit is more than offset by the increased thickness of the cover layer. These tests indicated that $n = 2$ is the optimum for tetrapod cover layers.

Damage or Safety-Factor Tests.—Because storm-wave trains contain waves higher than the significant height ($H_{1/3}$), it is important that rubble-mound breakwaters be designed so that they will not fail when subjected to waves with heights moderately larger than the selected design-wave height. Thus, quarry-stone armor units were subjected to tests in which wave heights were greater than the previously selected design-wave heights for the no-damage and no-overtopping criteria to obtain information concerning safety factors for rubble-mound breakwaters designed on the basis of Eq. 18. Results of these tests for quarry-stone armor units are presented in the form of a log-log plot in Fig. 5, with the stability number (N_s) as the ordinate, cot α as the abscissa, and the percentage of damage to the cover layer (D) as the parameter. The damage tests were conducted using 0.10-lb and 0.30-lb armor stones and relative depths of 0.10 and 0.25. The solid line AB in Fig. 5 is the same as line AB in Fig. 4; that is, it is the approximate best-fit line through the data points for the no-damage and no-overtopping criteria. The dashed lines in Fig. 5 were drawn parallel to line AB through data points delineating approximate ranges of percentages of damage to the cover layer. Although the dashed lines represent only rough approximations of the amounts of damage obtained for the different wave heights, it is believed that, considering the nature of the tests and the significance of the damage parameter, they reflect the test results with sufficient accuracy for the immediate needs of the design engineer.

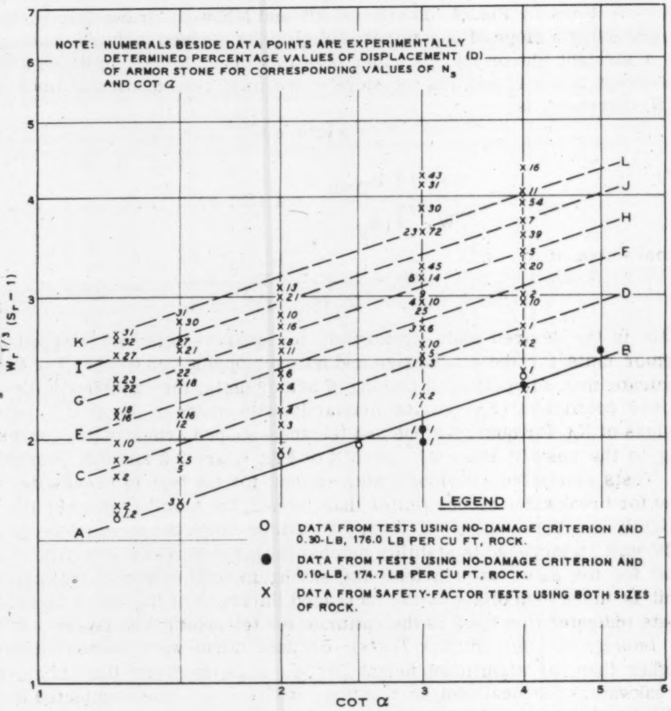


FIG. 5.—STABILITY OF QUARRY-STONE COVER LAYERS: N_S AS A FUNCTION OF α AND D

The form of the equation for the dashed lines in Fig. 5 is the same as that of lines AB and MN of Fig. 4. Therefore, the general formula for stability of quarry-stone armor units, for $H \geqq H_{D=0}$, is

$$W_r = \frac{\gamma_r \; H^3}{K_D \left(S_r - 1\right)^3 \; \cot \alpha} \quad \ldots\ldots\ldots\ldots\ldots (19)$$

in which K_D is the experimentally determined damage coefficient, and H is the corresponding wave height. Table 3 shows values of D, $H/H_{D=0}$, and K_D corresponding to the various lines in Fig. 5.

In this tabulation, the amounts of damage to the test sections are given in terms of percentages of the armor units removed from the cover layer. In the damage tests the breakwater sections were of the type shown in Fig. 3(a), with both the crown height above still-water level and the maximum distance below still-water level to which the armor units extended being equal in magnitude to the previously determined design-wave height. Thus, the percentages of damage for these tests are considerably smaller than the corresponding percentages of damage that would obtain for breakwaters of the type shown in Fig. 3(b), other conditions being equal. In the Fig. 3(b) breakwater section, the volume of the cover layer is smaller than that shown in Fig. 3(a), consequently, for equal amounts of damage to the cover layer, the percentage of damage is proportionally larger for the cover layer of smaller volume.

Comprehensive tests to determine the amount of damage to tetrapod cover layers as a function of $H/H_{D=0}$ have not been conducted. However, preliminary tests of tetrapods in which waves larger than $H_{D=0}$ were used indicate that the limit of stability of tetrapod armor units, with n = 2, is reached when the ratio $H/H_{D=0}$ becomes equal to approximately 1.2. For values of $H/H_{D=0}$ slightly larger than 1.3, failure of the tetrapod cover layer occurs. It is believed, therefore, that a value of K_Δ of 8.3, which corresponds approximately to the lower envelope of data points for tetrapods in Fig. 4, line M'N', should be used for design of tetrapod cover layers until more quantitative information is available concerning safety factors for tetrapod armor units.

TABLE 3.—EXPERIMENTALLY DETERMINED DAMAGE COEFFICIENTS FOR QUARRY-STONE ARMOR UNITS

Line (1)	Range of D, in percentage (2)	$H/H_{D=0}$ (3)	K_D (4)
AB	0-1	1.00	3.2
CD	1-5	1.18	5.1
EF	5-15	1.33	7.2
GH	10-20	1.45	9.5
IJ	15-40	1.60	12.8
KL	30-60	1.72	15.9

It is emphasized that the wave heights in Eqs. 18 and 19 are the selected significant waves that occur at the position of a proposed breakwater before the breakwater is constructed, and not the heights of waves moving up, or breaking on, a breakwater slope. Also, it is pointed out that the angle (α) in these equations is the angle of the breakwater slope as first constructed, and not the angle of the breakwater slope after the breakwater has been stabilized by waves of height H.

SUPPLEMENTARY DESIGN DATA

Wave Run-Up.—The primary function of breakwaters is to provide adequate protection from wave action in selected harbor areas. Consequently, overtopping usually can be tolerated only if it is negligible or does not exceed allowable limits as determined by the type of harbor and the use for which different areas in the harbor are designed. There is considerable experimental data in the literature concerning wave run-up on paved slopes, beach slopes, and shore-line structures such as seawalls (11),(12),(13),(14), and a theoretical method of computing run-up on smooth, impervious slopes by Miche (15), has been noted by Bruun (16). However, comparatively little run-up data are available for structures with slopes as rough and porous as rubble-mound breakwaters.

Although limited in scope, the small-scale tests of wave run-up on sloping structures conducted by Granthem (17) provide some information on this subject. Granthem's tests were conducted in a manner that approximated the action of waves on rubble-mound breakwaters. Although derivation of a theoretical basis for interpretation and correlation of test data was not attempted, it is believed that the important parameters suggested by Granthem's tests can be used to correlate data obtained in the present testing program. Granthem concluded from the results of his tests that the primary variables affecting wave run-up are the wave steepness (H/λ), the relative depth (d/λ), the angle of the seaside slope (α), and the porosity of the structure (P).

Hydraulic roughness of the slope surface and the angle of obliquity of wave attack (β) are also believed to affect wave run-up. The hydraulic roughness of a breakwater slope is difficult to define quantitatively, however, for the quarry-stone armor units placed pell-mell, such as those used in this investigation, the average thickness of one layer of armor units should provide an approximate measure of this variable. Thus, correlation of the run-up data for rubble-mound breakwaters may be accomplished by the functional relationship

$$R/H = f(\alpha,\ H/\lambda,\ P,\ d/\lambda,\ r,\ \beta) \ldots\ldots\ldots\ldots (20)$$

The percentage of voids in the quarry-stone cover layers of the breakwaters tested was essentially constant, and the angle of wave obliquity was 0 deg. Therefore, for the tests completed to date, Eq. 20 reduces to

$$R/H = f(\alpha,\ H/\lambda,\ d/\lambda,\ r) \ldots\ldots\ldots\ldots (21)$$

Wave run-up data were obtained by visual observation. The average of five individual readings was recorded for each size wave used in the testing of each section. Each of the five individual readings represented the average run-up for a wave train consisting of from 10 waves to 15 waves.

Results of the run-up observations are presented graphically in Figs. 6 through 8. These data show that the wave run-up factor (R/H) is a function of breakwater slope, wave steepness and, to some extent, the hydraulic roughness of the breakwater surface. The effects of relative depth are obscured by the wide range of scatter in the observed values of run-up, that is attributed to difficulties in defining and observing the extent of run-up on a rough, porous, sloping surface, and the complexity of the phenomenon of wave motion on rubble-mound slopes. The range of scatter should be even larger for wave run-up measurements on full-scale structures. Therefore, it is believed that the upper limits of the envelopes of data points, indicated by the solid lines in Figs. 6 through 8, should be used in selecting design crown elevations when overtopping of a proposed rubble-mound breakwater cannot be tolerated.

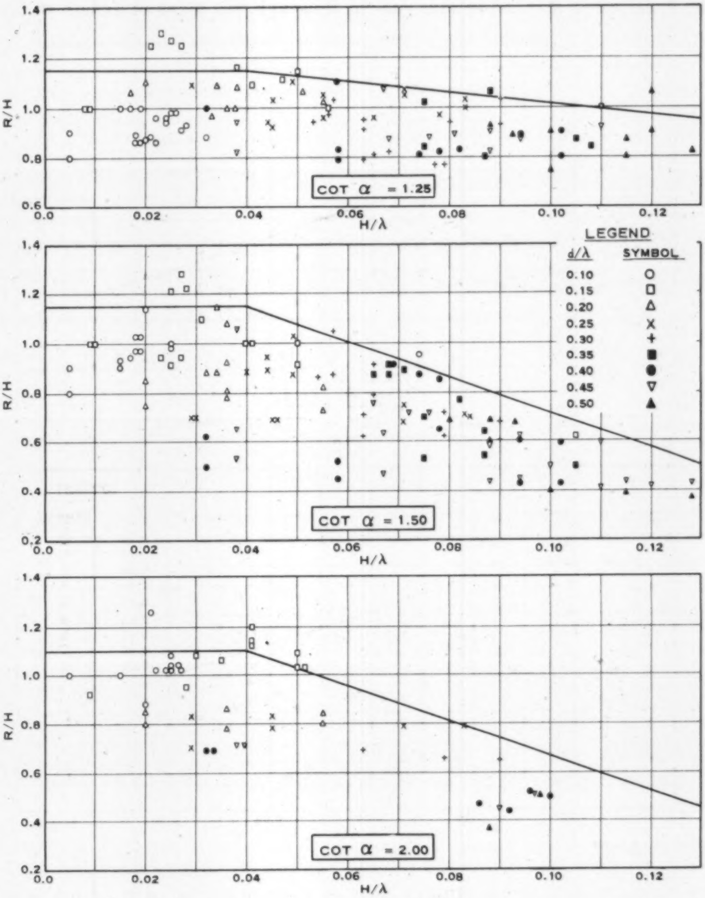
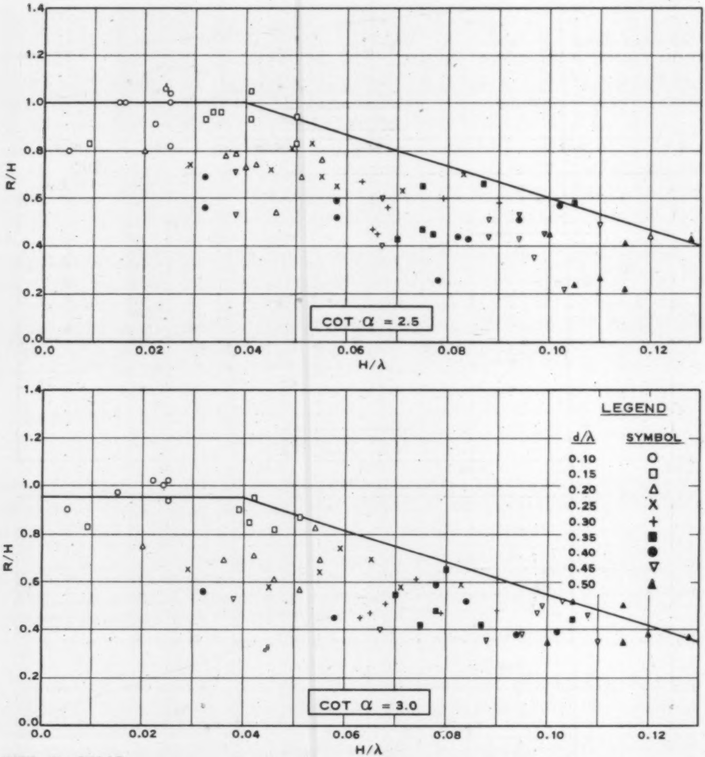


FIG. 6.—WAVE RUN-UP ON RUBBLE-MOUND BREAKWATERS: $R/H = f\,(H/\lambda,\ d/\lambda)$

The test data show that breakwater slope and wave steepness are primary variables affecting wave run-up on porous rubble-mound breakwaters of the type tested. Within the range of test conditions used to date, R/H decreases when either cot α or H/λ is increased.

The tests were not designed to study the effects of the hydraulic roughness of the breakwater surface on wave run-up. However, two sizes of armor stones were used in tests of sections with side slopes of 1-on-4 and 1-on-5. Results show that size of stones does not affect wave run-up on a slope of 1-on-4. For the 1-on-5 slope, however, the run-up factors for the smoother surface, that is, the slope composed of 0.10-lb armor stones, averaged approximately 20% greater than the corresponding run-up factors for the slope composed of



NOTE: W_r = 0.30 LB

FIG. 7.—WAVE RUN-UP ON RUBBLE-MOUND BREAKWATERS: $R/H = f(H/\lambda, d/\lambda)$

0.30-lb stones. This can probably be explained by the fact that waves tend to break more readily on flatter slopes, and as the breaking waves rush up the slope, the depth of flow decreases, resulting in a greater percentage of energy loss for the rougher surface. Also, the flatter slopes provide a greater distance over which the losses of energy may occur. However, these tests are

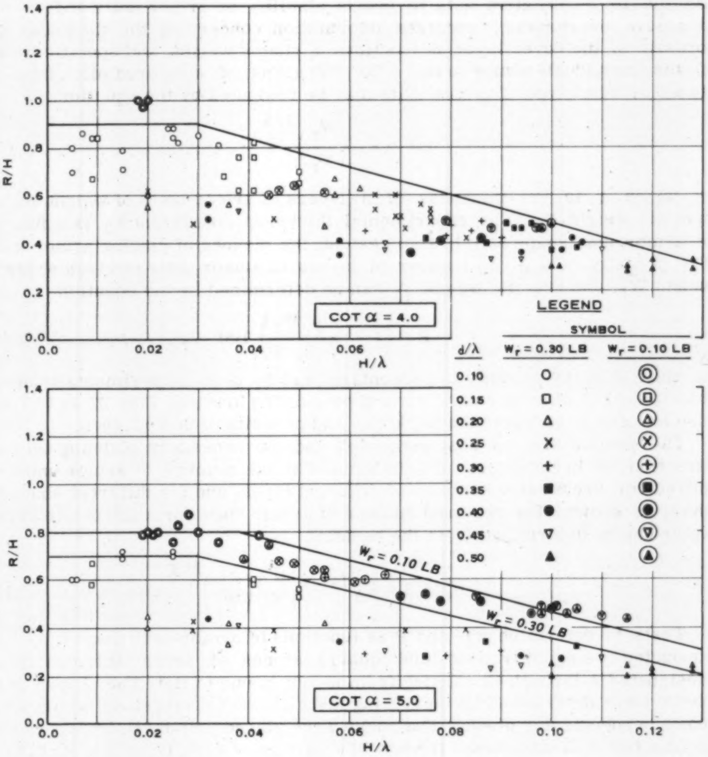


FIG. 8.—WAVE RUN-UP ON RUBBLE-MOUND BREAKWATERS: $R/H = f(H/\lambda, d/\lambda, r)$

not sufficient to determine fully and accurately the effects of hydraulic roughness on wave run-up, and additional tests are therefore necessary.

A qualitative measure of the effects of porosity can be obtained by comparing the results of the present tests with results of those conducted at the Waterways Experiment Station during 1954 and 1955 in an investigation of wave run-up on Lake Okeechobee levee slopes (18). The porosity of the armor-stone cover layers used in the present tests averages about 41%. Levee slopes used in the Lake Okeechobee tests were smooth and impervious. The comparison of the results of these two sets of tests showed that the run-up factor for the smooth impervious slopes averages about twice that obtained for the comparatively rough, porous slopes used in the tests of rubble-mound breakwaters.

Thickness and Porosity of Cover Layers.—Breakwater design requires, in addition to quantitative data to insure stability of armor units and prevent excessive overtopping, accurate information concerning the thickness and porosity of the cover layer as functions of shape, weight, and specific weight of the individual armor units. The thickness of a layered pile of quarry stones or other type of armor units may be computed by the equation

$$r = n\, k_\Delta \left(\frac{W_r}{\gamma_r}\right)^{1/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

in which r is the thickness of n layers of armor units of weight W_r and specific weight γ_r. The experimental thickness coefficient k_Δ is a function of armor-unit shape and, to some extent, the manner of placing armor units. The porosity of a given number of layers of armor units of given shape Δ, weight W_r, and specific weight γ_r can be determined by the equation

$$P = \left(1 - \frac{N_r\ W_r}{A\ \gamma_r\ r}\right) 100 \quad \ldots\ldots\ldots\ldots\ldots (23)$$

in which P is the porosity in percentage, and N_r is the experimentally determined number of armor units for a given surface area, A. Eqs. 22 and 23 may also be used to estimate the thickness and porosity of underlayers.

The preparation of cost estimates and the necessary planning for construction of breakwaters are facilitated if the number of armor units required for breakwater sections of different types, and for different shapes of units, is known. The required number of armor units for a full-scale breakwater can be determined from the equation

$$N_r = A\, n\, k_\Delta \left(1 - \frac{P}{100}\right)\left(\frac{\gamma_r}{W_r}\right)^{2/3} \quad \ldots\ldots\ldots\ldots (24)$$

Tests to determine k_Δ and P as functions of armor-unit shape have been conducted using tetrapods and quarry stones of seven different shapes (designated A through G) varying from nearly round to flat. The shapes of the rocks were determined by measuring their average dimensions in three mutually perpendicular planes. The rocks were placed pell-mell, by layers, in a square box 2 ft wide and 1 ft high. The surface of each layer was sounded to determine its average thickness, and the number of rocks required to form each layer was counted. The thickness coefficient (k_Δ) and the porosity of the rock layers (P) were then computed by means of Eqs. 22 and 23.

Thickness and porosity data were obtained for one, two, three, and four layers of each shape of rock. Individual stones of each type having approximately the same weights and shapes were selected. The rocks varied in weight from 0.12 lb to 0.46 lb, and had an average specific weight of 176.0 pcf. The manner of placing the rounder rocks (shapes A, B, and C) corresponds to pell-mell construction. For the more elongated rocks (shapes D to G), the manner of placement corresponded roughly to masonry-type construction,

TABLE 4.—SHAPE AND POROSITY CHARACTERISTICS OF QUARRY-STONE ARMOR UNITS

Number of Layers, n (1)	Characteristics of Stone			
	x/z (2)	y/z (3)	k_{Δ} (4)	P, % (5)
Stone Shape A				
1	1.5	1.2	0.95	38
2	1.5	1.2	0.95	40
3	1.5	1.2	0.93	41
4	1.5	1.2	0.91	38
			Avg 0.94	39
Stone Shape B				
1	1.6	1.3	0.95	44
2	1.6	1.3	0.93	41
3	1.6	1.3	0.92	40
4	1.6	1.3	0.93	40
			Avg 0.93	41
Stone Shape C				
1	1.6	1.3	0.92	40
2	1.6	1.3	0.91	39
3	1.6	1.3	0.92	42
4	1.6	1.3	0.91	41
			Avg 0.92	40
Stone Shape D				
1	1.7	1.4	0.89	43
2	1.7	1.4	0.92	45
3	1.7	1.4	0.91	43
4	1.7	1.4	0.89	42
			Avg 0.90	43
Stone Shape E				
1	2.2	1.5	0.81	43
2	2.2	1.5	0.81	42
3	2.2	1.5	0.81	42
4	2.2	1.5	0.80	41
			Avg 0.81	42
Stone Shape F				
1	2.6	1.5	0.76	46
2	2.6	1.5	0.77	47
3	2.6	1.5	0.75	45
4	2.6	1.5	0.75	45
			Avg 0.76	46
Stone Shape G				
1	3.3	2.5	0.62	49
2	3.3	2.5	0.65	45
3	3.3	2.5	0.66	48
4	3.3	2.5	0.64	46
			Avg 0.64	47

with the largest dimension of the rock parallel to the breakwater slope. This manner of placing elongated stones was used to determine the effects of shape factor on the coefficients k_Δ and P, and is not recommended for full-scale breakwater construction.

Tests of tetrapod armor units were made using the 2-ft-sq box as described previously, with the units placed pell-mell, two layers thick. Tests of tetrapods were also made in which the two layers were placed in the cover layer of a breakwater test section in a more dense and geometrical pattern.

The results of tests to determine the thickness coefficient (k_Δ) and the porosity (P) for the different shapes of quarry-stone armor units are shown in Table 4. Both k_Δ and P vary with shape of the stones; neither, however, vary with the number of stone layers (n). The thickness coefficient has an average value of 0.94 for the shape A (nearly round) armor stones, and decreases as the shape of the stones becomes flatter and more elongated, to an average value of 0.64 for the shape G stones. Porosity increases as the shape of stones becomes flatter and more elongated. The average values of P for the shape A and shape G stones are 39% and 47%, respectively.

TABLE 5.—SHAPE AND POROSITY CHARACTERISTICS FOR TETRAPOD ARMOR UNITS

n	k_Δ	P, %	Placement	Reference
2	1.02	49	Pell-mell	Danel
2	1.06	43	Geometrical	WES data - 30 tests
2	1.00	52	Pell-mell	WES data - 5 tests
2	1.13	46	Pell-mell below swl, geometrical above swl	Hudson and Jackson
3	1.02	46		
4	0.96	46		

The results of tests to determine values of k_Δ and P for tetrapod armor units are shown in Table 5. For tetrapods placed geometrically, two layers thick, average values of k_Δ and P are 1.06 and 43%, respectively. For tetrapods placed pell-mell, two layers thick, the respective values are 1.00 and 52%. In addition, results of Danel (7) and Hudson and Jackson (9) are shown for tetrapods placed pell-mell and placed semi pell-mell, respectively. It is believed that values of k_Δ and P of 1.0 and 50% are representative of the conditions that would obtain when placing tetrapods in two layers to form cover layers of full-scale breakwaters.

CONCLUSIONS

The following is concluded from the results of tests completed (as of 1959) on small-scale rubble-mound breakwaters with quarry-stone and tetrapod-shaped armor units:

1. Iribarren's formula is not sufficiently accurate to be used in designing rubble-mound breakwaters unless it is used in conjunction with values of the experimentally determined coefficient K′, as a function of breakwater slope, shape of armor unit, and the other important variables described herein.

2. Use of the Iribarren formula in correlating the stability-test data for rubble-mound breakwaters is not feasible, because the experimental coefficient K' varies appreciably with the coefficient of friction μ, and accurate values of the friction coefficient for the different types of armor units are very difficult to obtain.

3. The assumptions on which the analysis of the phenomenon of waves attacking a rubble-mound breakwater was based are sufficiently accurate for purposes of this investigation.

4. Results of the stability tests conducted for the no-damage and no-overtopping criteria are represented with sufficient accuracy by Eq. 18.

5. The amount of damage that will be done to a quarry-stone cover layer of the type tested by waves larger than the selected design wave can be estimated from the results of damage tests presented in this paper.

6. The safety factor for rubble-mound breakwaters with quarry-stone armor units and $n > 2$, designed in accordance with Eq. 18 using $K_\Delta = 3.2$, is adequate. However, in view of the fact that nature wave trains contain waves of heights as large as $1.6\,H_{1/3}$ approximately 1% of the time, compared with a corresponding value of $1.1\,H_{1/3}$ for the small-scale test waves, there is some doubt as to which of the various wave heights in natural wave trains should be selected as the design wave.

7. Eq. 18, with a value of 8.3 for K_Δ, can be used to design tetrapod cover layers for rubble-mound breakwaters. However, because preliminary tests have indicated that tetrapod cover layers with $n = 2$ are damaged appreciably by waves slightly larger than $1.3\,H_{D=0}$, it is recommended that design-wave heights for breakwaters having this type of cover layer be selected with caution.

8. For the conditions tested, in which the H/d ratio was comparatively small, the stability of rubble-mound breakwaters is not appreciably affected by variations in the d/λ and H/λ ratios. However, special stability tests concerning a breakwater at Nawiliwili Harbor, Kauai, T. H. (19), where the H/d ratio is critical and waves break directly on the breakwater slope, showed that the ratios H/λ and d/λ are important variables for these conditions.

9. Two layers of armor units are optimum for tetrapod cover layers.

10. Breakwater slope ($\tan \alpha$) and wave steepness (H/λ) are the primary variables affecting wave run-up on rubble-mound breakwaters where the H/d ratio is sufficiently large so that breaking waves do not occur on or seaward of the breakwater slope. Wave run-up decreases when values of either H/λ and $\cot \alpha$ are increased.

11. The thickness of cover layers and the number of armor units required to cover exposed slopes of rubble-mound breakwaters can be determined by the Eqs. 22 and 24. Conservative values of k_Δ and P for selected quarry-stone armor units placed pell-mell are 1.0 and 40%, respectively. Corresponding values of k_Δ and P for tetrapods are 1.0 and 50%, respectively.

ADDITIONAL TESTS

Tests being conducted (April, 1959) at the Waterways Experiment Station to determine the relative efficiencies of quarry-stone, tetrapod, tribar, tetrahedron, and other special-shape armor units. Tribars (20), (21) were developed by R. Q. Palmer of the U. S. Army Engineer District, Honolulu, T. H.

As of this date (April, 1959), test results obtained at the Waterways Experiment Station indicate that, with n = 2; (a) tetrahedrons are inferior to both tetrapods and tribars with respect to stability, (b) tribars are slightly better than tetrapods with respect to stability, (c) a tribar cover layer has a slightly higher porosity than a tetrapod cover layer, and (d) a smaller number of tribars are required for a two-layer cover for rubble-mound breakwaters. Also, it has been determined that tribars can be placed as a one-layer unit above still-water level in such a way that the stability provided is considerably greater than the stability provided by two layers of either tetrapods or tribars.

ACKNOWLEDGMENTS

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APPENDIX I.—BIBLIOGRAPHY FOR RUBBLE-MOUND BREAKWATERS

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APPENDIX II.—NOTATION

A = Surface area, square feet;

a = coefficient in Eqs. 17, 18, and 19;

b = exponent in Eq. 17;

C = coefficient;

D = damage parameter, percent;

d = water depth, feet;

F = force, pounds;

f = reads "function of";

g = acceleration due to gravity, feet per second squared;

H = wave height, i.e., the vertical distance from trough to crest, measured at the location of a proposed breakwater, feet;

h = height of breakwater crown above swl, feet;

K = coefficient in new breakwater stability formula for conditions of no damage and no overtopping, varies with shape of armor unit;

K' = coefficient in modified Iribarren formula;

K_D = coefficient in new breakwater stability formula, varies with percentage of damage to cover layer for a given shape armor unit;

k = coefficient;

l = characteristic linear dimension of armor unit, feet;

m = width of breakwater crown, feet;

N = number;

n = number of layers of armor units;

P = porosity of cover layer, percent;

R = wave run-up, vertically above swl, feet;

R = Reynolds number;

r = thickness of cover layer, measured perpendicular to slope of breakwater face, feet;

S = specific gravity, for example, $S_r = \gamma_r/\gamma_w$;

T = wave period, seconds;

t = time, seconds;

V = velocity, feet per second;

W = weight, pounds;

W'_r = buoyant weight of armor unit, pounds;

x = abscissa;

y = ordinate;

z = vertical distance, measured positively upward from swl, feet;

α = angle of breakwater slope, measured from horizontal degrees;

β = angle of obliquity of wave attack, degrees, for example, when wave crest is parallel to breakwater alignment, $\beta = 0$;

γ = specific weight, pounds per cubic foot;

Δ = shape of armor unit;

λ = wave length, feet;

μ = coefficient of friction;

σ = angle of beach slope, measured from horizontal, degrees;

ϕ = angle of repose of armor units, degrees;

$\cot \alpha$ = reciprocal of breakwater slope;

$\tan \alpha$ = breakwater slope;

∂ = partial differential symbol;

R/H = wave run-up factor;

$d/$ = relative depth;

$H/$ = wave steepness;

H/d = relative height;

$\frac{1}{V^2} \frac{\partial V}{\partial t}$ = form of Iversen's modulus; and

swl = still-water level.

SUBSCRIPTS

a = refers to area;

b = refers to break-wave condition;

D = refers to damage of cover layer;

d = refers to drag;

m = refers to inertia;

q = refers to total;

r = refers to armor unit;

s = refers to stability;

v = refers to volume;

w = refers to water; and

Δ = refers to slope factor.

DISCUSSION

JOSÉ REIS DE CARVALHO[3] and **DANIEL VERA-CRUZ**,[4] **A.M. ASCE**.—The writers have followed the extensive and valuable work done in the Waterways Experiment Station on breakwater stability, of which Mr. Hudson's paper is a part. The paper shows the new trend in the study program of that station due to the difficulty in obtaining the exact value of the friction coefficient in Iribarren's formula, the accurate determination of which was the purpose of the former program. In spite of the considerable work already carried out, much remains to be done before the design engineer will be able to achieve an optimum design. For that purpose, the design engineer must not only know how to design the armor cover layer, but also, especially when dealing with great depths, to design the underwater slopes below the lower level of the cover layer. He must also know how to solve the problems involved in the singular points of breakwaters, such as curves and heads. When dealing with shallow depths, the situation is very different from that prevailing at great depths, especially on sandy bottoms when the waves break just before or on the breakwater toe. In this case, even when measures are taken against the setting down of the toe, the increased specific gravity of the water resulting from the great amounts of sand stirred up, must be taken into account. Believing that this is an extremely important point, the writers suggest that, although the experimental conditions during the tests are presented in the text, it would be preferable to stress in the figures of the paper summing up the experimental results that these, due to the test conditions, have a restricted field of application.

The Formula Proposed by the Author.—The author considers that the armor units acted on by the wave are subjected to two principal forces: a drag force and an inertia force. It is the opinion of the writers that the inertia force is very important and that, if the influence of the term $\frac{\partial V}{\partial t}$ could be investigated, a better understanding of the phenomena occurring in a breakwater would result. However, it being difficult to determine this influence, the author includes this term in the same coefficient that involves the drag coefficient, the virtual-mass coefficient, and the shape of the units.

To establish the incipient instability condition, Hudson makes the resultant of the two forces developed by the waves equal to the buoyant weight of the armor unit. He, however, takes only the magnitude of those forces into consideration, neglecting their directions. It is the opinion of the writers that an effort must be made to express the vectorial character of the forces that is supposed to be all the more important the steeper the slopes. With these simplifications, the proposed formula is:

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$$\frac{\gamma_r^{1/3}}{\left(S_r-1\right)W_r^{1/3}} = f\left(\alpha, \frac{H}{\lambda}, \frac{d}{\lambda}, C_d, C_n, K_a, K_v, \ \ldots\right) \ldots\ldots (25)$$

in which the first member is a dimensionless parameter and the symbols are those defined in the paper.

For the test conditions, the author found,

$$\frac{\gamma_r^{1/3} H}{W_r^{1/3}\left(S_r-1\right)} = a\,(\cot\alpha)^{1/3} = \left(K_\Delta \cot\alpha\right)^{1/3} \ \ldots\ldots\ldots (26)$$

in which K_Δ is the shape coefficient, numerical values of which are presented for quarry-stone and tetrapod-shaped units.

Criteria of Stability.—One of the chief advantages of Hudson's paper lies, no doubt, in the adoption of numerical, consequently well defined, stability criteria. In all the experimental studies so far carried out, both at European laboratories and in those previously described by the Waterways Experiment Station, the criterion left an important part of the experimenter's personal judgment in regard its definition and above all its appreciation. It has thus been impossible to make an accurate comparison of the very numerous results already obtained by the different laboratories in which the subject was studied.

The writers believe, nevertheless, that a real effort should be attempted to improve the stability criterion established by the author, for the no-damage condition at least, so that the results can be generalized for use in the design of similar structures.

In fact, the geometrical characteristics of the breakwater sections used in the tests on the quarry-stone armor units and the tetrapod armor units being considerably dissimilar, the results obtained are not comparable, because, as Hudson states, the percentage of damage is proportionally higher for the cover layer of smaller volume. It seems that the most adequate criterion for the no-damage condition should be one in which only the active portion of the breakwater and one or two layers of the protective cover were taken into account.

As regards the tests for determination of the damage or safety factor, it seems that the results obtained apply only to the types of breakwater sections tested. It suffices to point out that, according to Hudson, failure of the tetrapod cover layer occurs for values of $H/H_{D=0}$ slightly exceeding 1.3, whereas for identical values of $H/H_{D=0}$ in the quarry-stone armor unit tests, only 5% to 15% damages were observed. Consequently, we believe that safety factors should be determined for each particular case, as it is very difficult, if not impossible, to find a criterion of stability in which the multiple causes conducive to damage in breakwater subjected to waves with a $H > H_{D=0}$ could be considered.

Iribarren's Formula.—In the deduction of his formula, Iribarren considers as the principal resistant force the one resulting from friction between blocks. His formula is Hudson's Eq. 1.

With regard to μ, Iribarren says that it will be very close to the tangent of the natural slope of the layer-units. According to Iribarren, μ is greater than 1, but he recommends that one use $\mu = 1$, that would introduce a certain safety factor. This point must be explained, and the writers need to present here the conditions near the quarry-stone breakwater, the observation of which permitted Iribarren to determine the coefficient to be used with quarry-stone breakwaters. Iribarren used the non-homogeneous expression of Eq. 1,

$$W_r = \frac{N H^3 S_r \mu^3}{(\mu \cos \alpha - \sin \alpha)^3 \left(S_r - 1\right)^3} \quad \ldots\ldots\ldots\ldots (27)$$

in which

$$N = K' \gamma_w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

The breakwater observed by Iribarren was of quarry stone with a sandy bottom, its water depth in high water was 4.5 m and the slope toe lay at the low-water level. The stones had a weight of 3 metric tons, $S_r = 2.65$, and the equilibrium slope was $\cot \alpha = 3.1$. Iribarren assumed that the height of the maximum wave was $H = 4.5$ m (water depth in high water) and $\mu = 1$. A value of N results equal to 0.015. If, according to Iribarren, one takes $\mu > 1$, a new value would result for N. Let us call this new value N'. It is easy to see that $N' > N$ and that, with such a value N' and the corresponding μ, no safety factor would have been introduced by taking $\mu = 1$ and the corresponding value of N. If now, according to Iribarren, we assume that N and N' are not variable with the slope, it can be seen by a simple computation, that $\mu = 1$ is an unsafe assumption when $\cot \alpha < 3.1$, and a safe one when $\cot \alpha > 3.1$. However, the safety factor introduced in these slopes varies with the slope, and this variation can be very important (for example, if the actual value is $\mu = 1.09$, the safety factor with $\mu = 1$ is about 1.1 if $\cot \alpha = 2$ and 2.2 if $\cot \alpha = 1.25$). The difficulty in defining with exactitude what is the natural slope, makes the use of Iribarren's formula too uncertain.

Besides, it is the opinion of the writers that, owing to the particular conditions near the quarry-stone breakwater observed by Iribarren (very small depths and sandy bottom), the generalization of the coefficient determined is very problematical.

Comparison Between Hudson's and Iribarren's Formulas.—Let us consider Eq. 26. We can give a similar expression to Iribarren's Eq. 1:

$$\frac{\gamma_r^{1/3} H}{W_r^{1/3}\left(S_r - 1\right)} = \frac{(\mu \cos \alpha - \sin \alpha)}{K'^{1/3} \mu} \quad \ldots\ldots\ldots\ldots (29)$$

Let $\left(\frac{1}{K'}\right)^{1/3}$ = K, equating the two second members of Eqs. 26 and 29:

$$K \cos \alpha - \frac{K}{\mu} \sin \alpha = K_{\Delta}^{1/3} (\cot \alpha)^{1/3}$$

$$K (\cos \alpha - \frac{1}{\mu} \sin \alpha) = a (\cot \alpha)^{1/3}$$

$$K = \frac{a (\cot \alpha)^{1/3}}{\cos \alpha - \frac{1}{\mu} \sin \alpha} = a \frac{(\cot \alpha)^{1/3}}{\cos \alpha - \frac{1}{\mu} \sin \alpha}$$

$$K'^{1/3} = \frac{\cos \alpha - \frac{\sin \alpha}{\mu}}{a (\cot \alpha)^{1/3}}$$

$$K' = \frac{\left(\cos \alpha - \frac{\sin \alpha}{\mu}\right)^3}{K_{\Delta} \cot \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

This expression relates Iribarren's coefficient K' with Hudson's coefficient K_{Δ}. And, K_{Δ} being a constant for the test conditions, the preceding expression shows that Iribarren's coefficient is a function of the slope α.

Using Eq. 30 we can always suppose $\mu = 1$ when cot α is greater than one. By this means, the writers think that Iribarren's formula is susceptible of experimental verification, provided that its coefficient is not considered a constant.

Summing up, the writers think that the great merit of Hudson's coefficient lies in the important advantage of being constant, whatever the slope. It should not, however, be called a shape coefficient, as it necessarily involves the friction as well and is also dependent on the physical nature of the surface of the blocks.

WILLIAM H. BOOTH, F. ASCE.[5]—Due to the peculiar character of rubble structures and the phenomena of wave action thereon, it has been difficult, in the past, to arrive at designs that are both safe, from a structural standpoint, as well as being economical to construct. The many variables present in the phenomena of waves attacking a rubble structure greatly complicate the problem of analysis. The science of rubble breakwater structures is still in an early stage of development. Hudson has done a great service to the engineering profession in presenting the model data developed to date. However, much work remains to be done.

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Until recent years, the design of rubble-mound breakwaters was usually based on observed performance of like structures without any regard to the wave characteristics at the proposed location. As a result, several rubble-mound structures have been damaged and maintenance costs have been high.

The original purpose of the laboratory investigation was to provide data from which an efficient design of rubble-mound breakwaters could be selected for different conditions of use and wave attack. It was believed that the results of the investigation would allow the formulation of equations for more scientific designs. The original investigation has been expanded and the results to date are contained in the paper.

The new stability formula presented has been used to determine the weight of armour units for rubble-mound structures on several projects constructed by the Corps of Engineers. However, the values of the experimentally determined coefficient (K_Δ) should not be considered as final at this stage of the testing program. Additional model studies of rubble-mound breakwaters are being conducted in a wave tank capable of producing 6-ft waves. Information from these studies will provide data on scale effects. The wave tank is located at the Beach Erosion Board, Washington, D. C.

The weight of the tetrapods described by John E. Deignan,[6] F. ASCE, was determined from results of the small scale model study. His paper presents some of the problems encountered in the design and construction of projects of this nature. From a structural and hydraulic standpoint, the tetrapod is considered to have a desirable shape for good stability when subjected to wave forces. However, an indicated royalty cost in the United States of $3.00 per cu yd of tetrapod volume may limit its use to a small number of projects due to economy.

The tribar has been patented in the United States by R. Q. Palmer, M. ASCE. The patent provides for the manufacture and use by or for the Government for its purposes without royalty payment. Patent applications are pending in several countries. This shape of armour unit has been manufactured and used for the repair of the breakwater at Nawiliwili Harbor, Island of Kauai. The project is under the jurisdiction of the U. S. Army Engineer District, Honolulu. The weight of the units was determined under the same testing program. This breakwater is a rubble-mound structure about 2,150 ft in length that had been damaged by storm waves. Quarry rock of sufficient weight to insure stability of the breakwater, without excessively flat slopes, was not available. Because preliminary model studies indicated that the tribar was slightly better than the tetrapod, it was decided to prepare three alternate plans for reconstruction of the breakwater. One plan was designed to use only stone. Two other plans were designed using precast concrete shapes on the seaward slope. One of these called for the use of 20-ton tetrapods and the other called for the use of 17.8-ton tribars. The tribar armour facing as well as the tetrapod armour facing, requires a solid backing to present ravelling when subjected to design wave overtop the structure. Therefore, both plans provided for concrete caps. Bids for the reconstruction were opened by the Honolulu District Engineer on April 30, 1958. The low bid was on the tribar plan. Since completion of the project, the breakwater has been subjected to heavy wave action. Preliminary information available at this time (December, 1959) indicates minor shifting of the tribars down the slope. The maximum measured distance was about 9 in. This movement was expected due to the consolidation of the rubble-mound and the tribar cover layer.

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[6] "Breakwater at Crescent City, California," by John E. Deignan, Proceedings, ASCE, Vol. 85, No. WW3, September, 1959, p. 167.

It is desired to emphasize the point that weight of armour units, determined in accordance with experimental data presented, will not necessarily apply to the seaward end, or head, of breakwaters. The ends are subjected to severe wave action and must be strengthened accordingly. In 1958, storm waves moved tetrapods weighing approximately 33 tons each off the heads of the breakwaters at Kahului Harbor, Territory of Hawaii. The tetrapods on the head were of the same weight as those designed and placed on the seaward slope of the breakwater. The expanded laboratory investigation program includes tests for the treatment of the head of breakwaters. It is hoped the investigation can be continued in order to provide sufficient data to insure safe and economical designs of rubble structures.

FRANCIS B. SLICHTER, F. ASCE.[7]—Hudson's development of hydraulic model testing procedures and formulas for evaluating test results are significant contributions toward rationalizing the variables involved in breakwater design.

To be effective, a breakwater must accomplish its purpose of reducing the waves generated by severe storms to tolerable levels in the designated harbor area. Often, several alternative breakwater layouts can be developed that, although not equal in performance, may give acceptable results. Consequently, decision on the layout to be selected will depend on an economic analysis considering first cost, maintenance costs, and relative performance in protecting the harbor area. In fact, economic limitations can force the acceptance of a breakwater design that allows severe harbor disturbance or breakwater damage during critical storm periods.

Breakwater design is far removed from status as a science in which application of formulas will lead to an exact solution. In addition to gaps in knowledge of wave phenomena wherefrom we derive the "design wave," we lack finite information on the reaction of the breakwater armour units and of its core material and foundation under the complex hydraulic impact of the waves. When we adopt a design using armour units of pre-cast concrete, we must accept unknown limitations imposed by structural strength of fabricated units and durability of concrete (and reinforcing steel, if used) in seawater. Therefore, although the guide lines contributed by Hudson's work are valuable aids, the design of breakwaters must remain largely in an area dominated by sound judgment based on observations of past experience.

In an effort to augment its experience record on fabricated armour units, the Corps of Engineers has constructed a tetrapod armoured breakwater at Crescent City, Calif., and has used the tribar shape in reconstruction of the breakwater at Nawaliwili Harbor, Island of Kauai, Hawaii. The weights of individual armour units are 25 tons, and 17.8 tons, respectively. Observations over several years are anticipated before conclusive data can be assembled as design criteria.

The value of experience is demonstrated by that of the Corps of Engineers at Kahului Harbor, Island of Maui, Hawaii. At this harbor two breakwaters converge to form the harbor entrance. The design wave height that is in the general magnitude of 28 ft actually will overtop the breakwater. The breakwaters were repaired in 1952, with armour stone of 12-ton weight. In the storm of March 1954, the ends of both breakwaters were damaged to the extent that restoration was essential to continued harbor use. Past experience with breakwater heads subjected to wave attack from divergent sources has demonstrated a requirement for monolithic cast-in-place concrete blocks with weight of several hundred tons. However, funds in the amount needed for such head construction were not available, and it was decided to rebuild the heads with the same weight armour unit selected for the breakwater proper. Consequently, the 33-ton units of the tetrapod shape used on the breakwater were also placed around each head. In December 1958, the breakwater was subjected to a storm delivering waves of design magnitude. Examination, subsequent to the storm, revealed that the tetrapods had been stripped from the face of each breakwater head. Some of the 33-ton tetrapods had been moved over 100 ft into the interior of the harbor. In fairness to Neyrpic (Grenoble, France), inventors of the tetrapod, their representative had advised that tetrapods used in armouring the head should be of greater weight than those used on the breakwater proper.

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[7] Chf., Engrg. Div., Civ. Works, Office, Chf. of Engrs., Washington, D. C.

In view of the magnitude of the weight scale ratio between the author's model armour unit and the prototype (over 300,000 for the Kahului tetrapod), it is doubted that there is any practical design significance in abandoning the Iribarren formula with its troublesome friction coefficient in favor of Hudson's new formula. On the other hand, the derivation of Hudson's formula results in a valuable tool for his use in evaluation of his model test data.

In developing his formula for stability, he has considered that friction between armour units can be neglected. This assumption is questioned, in particular for fabricated shapes such as the tetrapod and tribar. It would appear that part of the large differences between stone and fabricated shapes in test values for K could be attributed to reactions caused by shear forces between units. Further research to explore the relative effects of unit-shape, porosity and reaction between units should contribute information useful in breakwater design.

Further work also is needed in developing criteria for armour shapes and in placement procedures for aid in design of the seaward end (head) of a breakwater.

LELAND B. JONES,[8] F. ASCE.—The advantages of the formula developed by Hudson are evident when one considers the uncertainties inherent in breakwater design. Use of the formula with the shape factor K_Δ, if selected with reasonable care, should lead to consistently better design than use of Iribarren's formula in which the accurate selection of the friction factor is both critical to accurate design and difficult to do, especially when manufactured shapes are used.

Because close control can be maintained in sizes and shapes of materials used for breakwater construction, testing of uniform materials leads to practicable design and reduces the number of variables that would otherwise require testing. Also, breakwaters are usually constructed in relatively shallow water, and it has not been necessary to investigate reduction in stability requirements very far below the water surface. For these reasons, rubble-mound breakwater investigations have been confined primarily to testing the stability of uniform size quarry stone and manufactured units at elevations within the range of wave action.

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[8] Civ. Engr., Chf. of Soils Sect., Corps of Engrs., Walla Walla, Wash.

When railroads, highways, and other structures are constructed along lakes and reservoirs it is often necessary to protect them against wave action, but it has not been considered economical to require uniform size materials or manufactured units for this purpose. Instead, riprap consisting of quarry-run rock having a gradation range is used. The selection of rock sizes and other riprap features has not always been very rational, and often they have been based on the engineer's judgment, supplemented by experience and the limited amount of available data. In the past some effort has been made to utilize breakwater design criteria for determining required rock sizes needed for graded riprap, but there has always been an uncertainty as to the effects of gradation and how to determine the significant rock sizes.

Development of flood control, navigation, and water power along the lower Columbia and Snake Rivers, in Oregon and Washington, makes a more precise knowledge of riprap requirements necessary. The Corps of Engineers has completed three dams on the lower Columbia River: Bonneville, The Dalles and McNary. Construction is just commencing (1960) on John Day Dam on the Columbia River, and Ice Harbor Dam on the lower Snake River is nearing completion. Three more dams are authorized for construction on the lower Snake River downstream of Lewiston, Idaho. All of these completed, under construction, and authorized projects have these things in common: all are in deep, narrow, steep-walled canyons; all have railroad and highway relocations that are or will be located along the edges of the reservoirs with high fills extending well below the water surface; and all are subject to relatively severe wave action. Riprap design for each of the completed projects was based primarily on Corps of Engineer's experience criteria, but required rock sizes were compared with W_r in Iribarren's formula, in which the assumed friction factor was 1.05. There was some question as to the validity of this assumption because W_r might represent a stone-size smaller than maximum size, water pressures inside the riprap might assume greater importance than in open, one-size stone, and the friction factor might be in error. It was found by measurement that dumped rock fill generally assumed outside slopes of 1 vertical on 1.3 or 1.4 horizontal, that could mean that a friction factor of 0.75 or 0.8 might be more reasonable. If the friction value were 0.75 the stone weight would be increased four times for a 1-on-2 slope.

Completion of Bonneville, The Dalles, and McNary Dams has given the Corps of Engineers, highway departments, and railroad companies with relocations along the reservoirs opportunity to observe the effects of wave action. It is obvious now that additional study is required for adequate and economical riprap design. More than 100 miles of riprap have been provided along the completed projects, and several hundred miles of riprap will be required along projects under construction and authorized. Inadequacies and overdesign can result in substantial extra cost here, where they would not be particularly important on smaller jobs.

As a result, the Waterways Experiment Station is investigating the following features of riprap design and other protection against wave action. These tests are being made for the Walla Walla district, Corps of Engineers, under the direct supervision of the writer.

1. Effects of wave heights and wave periods on graded riprap at the water surface and at variable depths below water surface;

2. necessary depth of riprap toe below water surface when embankment is rock fill;

3. necessary depth of riprap or rock fill toe below water surface when embankment is gravel;

4. height of wave ride-up and depth of wave rundown for graded riprap;

5. limited tests for variable riprap slopes; and

6. comparative effects of wave action on riprap for angles of attack of 30°, 60°, and 90°.

While the testing program has not progressed to the point at which results can be published, it has produced some interesting trends, as suspected, and these trends may become important factors in riprap design. For example:

a. The value of W_r for quarry stone, as determined from the author's formula, was compared with results of model tests for graded riprap. Preliminary data indicate the riprap stone size comparable to W_r may be in the order of 20% by weight or 50% by size of the maximum size stone. This was based on tests for well graded riprap where the minimum size stone was about 6% by weight of the maximum size. On this basis, it might be necessary to require more uniform stone sizes where wave action is severe and available stone sizes tend to be small, or where the volume of material required for graded riprap would make the cost excessive.

b. Tests on river gravel having 2-in maximum size and underwater slope of 1 vertical on 2 horizontal indicate that gravel fill may require riprap protection for 10 ft below water surface for wave heights in the order of 4 ft or 5 ft, and for about 30 ft below for wave heights in the order of 8 ft. The wave period appears to have considerable effect on stability of the gravel. It is interesting to note that the gravel, when tested for damage by wave action at water surface, resisted wave heights of about 0.5 ft.

Because tests are incomplete, it is not possible to present firm conclusions. However, it is believed that the work being performed on riprap is of general interest. These studies are, of necessity, geared to problems and materials typical of the Columbia and Snake River areas, but it is expected that significant data will be obtained to be of value on other related work. It will also be a starting point for future expansion of riprap testing.

OMAR J. LILLEVANG,[9] M. ASCE.—The engineer who struggles to keep abreast of the growth of knowledge in the field of harbor and coastal engineering can find valuable assistance in Hudson's paper. The bibliography refers to information that is found in only a few libraries and not many of the references include discussions by reviewers or supplementary contributions. To the reader who finds too little time for study, the discussion by others of the work of technical authors is a great aid to quick evaluation. Wider distribution of information on new work and ideas is needed; it is appreciated that work such as the present paper, originally given limited distribution by the Waterways Experiment Station, is published in the journals of the Society.

Design waves for rubble-mound structures may not be the same for stone size determination as for concerns with uprush of the seas. Where wave statistics are available for a problem site the record normally presents frequency of occurrence of the "Significant" waves and periods, the term being a precise one relating to the average of the highest one-third of the waves in a sample. Other averages within the sample are related by Hudson, and, in 1952, R. R. Putz[10] published an excerpt from an actual wave record that graphically shows characteristic variations. Fig. 9 is Putz' illustration, and from it one can clearly appreciate the uncertainty of what may come next in any wave series.

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[9] Asst. Engr., Leeds, Hill and Jewett, Los Angeles, Calif.

If the major waves were always tightly grouped, any heavy overtopping by uprush on seawalls would be an extremely burdensome problem of water disposal. Some risk can prudently be taken in most instances that the highest waves do not ordinarily group themselves within the series, and the structure design may, therefore, often be related to some lower average wave. Overtopping by the wave uprush doesn't necessarily mean the design is a failure. Accepting overtopping and providing for it in the design may be the best economic choice in many instances. Fig. 10 shows three basic variations of a reference cross section that may be considered in treating with the wave run-up question. Hudson's work provides excellent tools for raising the crest of the structure or for flattening the seaward slope to minimize or eliminate overtopping. It is apparent that if one would consider making the crest wider than the minimum width dictated by construction procedures, and drain off the cresting water through the armor voids, then data on the volume of water at any elevation in an uprushing wave would be most helpful. This need for knowledge of volume versus elevation becomes particularly acute if the structure is a seawall of normal crest width, and gutters or storm drains must be proportioned for the area landward of the seawall. If such information exists, its presentation should be sought; if not, it is an area of investigation which could very beneficially be undertaken in one of the research centers active in this field.

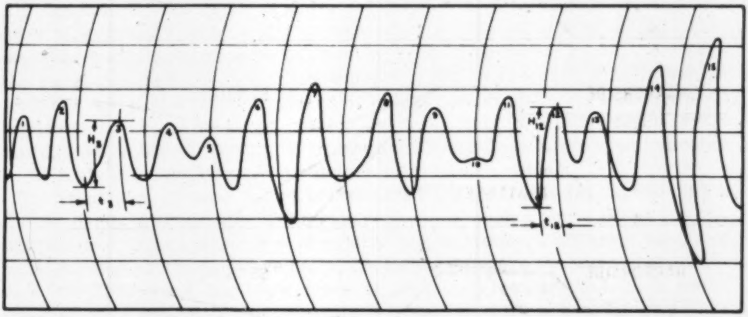

FIG. 9.—TYPICAL SECTION OF A WAVE RECORD

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[10] "Statistical Distribution for Ocean Waves," by R. R. Putz, Transactions, AGU, Vol. 33, No. 5, p. 685.

Armor stone might be selected of a size to completely resist the maximum wave in a train of waves, but something less is usually acceptable. Armor stone in well-built and successfully maintained breakwaters and seawalls are not often as large as would be indicated to be necessary from application of formulas that neglect interaction between stones (other than friction), or consider the interaction to be a safety factor. "Well-built," in this writer's view, requires that armor stone be placed with care, not dumped, and that above the low water line the individual pieces be selected for shape of fit to other stones and laid in juxtaposition with them and in an attitude of optimum stability. Wedging may then be anticipated as the whole structure accepts wave attack, and settles into a coherent unit. Lacking such careful selection and placing of armor stones, the wave-affected slope should be flattened so only friction is relied on, if maintenance is not to be excessively costly soon after the first heavy storms. Obviously, the variability of dimensions of stones of a given weight, their angularity or blockiness, will affect the degree of stability achieved by careful placement. In essence this is recognized in Hudson's use of a "Shape Factor." It would be useful if in the author's closing discussion some added data to illustrate the Shape Factor were included.

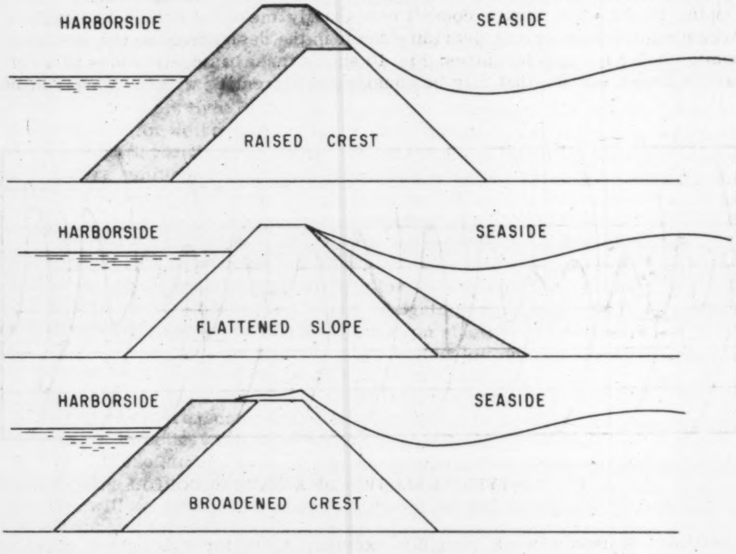

FIG. 10.—ALTERNATIVE TREATMENTS FOR WAVE RUNUP

The writer is intrigued by Hudson's statement that the effect of nonbreaking wave forces is comparable with that of the forces of breaking waves, though the wave motions are different in characteristics. An elaboration of this statement would be most interesting. Would this view lead one to rational design procedures in determining underwater slopes and stone sizes? For that unseen portion of rubble structures little has been found.

Presumably the tests, under way in the Beach Erosion Board's giant wave flume, on stability of very much larger stones in rubble-mound structures will evaluate the significance of scale effect in Hudson's experiments. As a follow-up of Hudson's paper, it is hoped the results of the large-scale work and its relationship to the small-scale will be the subject of a paper. The subject is worthy of the wider readership.

THORNDIKE SAVILLE, JR.,[11] **M. ASCE.**—The author has ably presented a quantity of useful data leading to continued progress toward a sounder basis for stability and economic design of rubble-mound structures. In the course of the extensive testing carried out, Hudson has also obtained a great deal of valuable data pertaining to wave run-up on rubble slopes. These data are presented in Figs. 8 through 10 in which relative run-up (R/H) has been plotted as a function of the wave steepness (H/λ) determined for the water depth at the structure toe. As the wave steepness for any particular wave train critically depends on the depth of water for which it is determined (varying by a factor of as much as 3 or 4, for ordinary waves), the plotted points have also been segregated according to relative depth (d/λ). However, the wave steepness range covered for each relative depth tested is generally small, and, as the author notes, the scatter induced by measurement difficulties and general complexity of action is large, so that the true effect of relative depth is obscured. Hudson has, accordingly, drawn a near-envelope curve, and indicates that he feels it should be used for determining design values.

Unfortunately, because values of both height (H) and length (λ) depend on the relative depth in which they are measured, use of such a curve under the assumption that it is independent of relative depth produces different values of run-up for any particular wave train, depending on where the wave characteristics are measured. An illustrative example is given in Table 6, in which values of wave run-up (R) as determined from Fig. 7 for a 1 on 2.5 slope are tabulated for a single wave train moving from deep water into shallow. The tabulations are made for arbitrarily selected water depths for which the wave characteristics are determined. The particular wave train chosen has a height and period of 8 ft and 6 sec in a 10-ft water depth. In making the computations, Fig. 7 was used as though it were completely independent of relative depth.

It will be noted that, in this case, predicted run-up of 5.9 ft is obtained if the wave characteristics, as measured in 10 ft of water, are used. However, if the designer had chosen to determine his wave characteristics at a depth of 25 ft instead, he would have obtained a predicted run-up of 6.8 ft. And if he were working with deep water characteristics, he would have determined a 7.8 ft predicted run-up. As the table shows, a different value of run-up will be determined for each relative depth value for which the wave characteristics may be measured. Actually, however, we know that a wave incident on a given structure must have only one value of run-up for a particular shore condition.

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[11] Asst. Chf., Research Div., Beach Erosion Bd., U. S. Army Corps of Engrs., Washington, D. C.

It would seem, therefore, that run-up curves such as these should be referred to a specific relative depth rather than being considered as independent of relative depth. Fortunately, there are readily available methods of determining heights and lengths for any depth, or relative depth, if the height and length are known at some particular depth. Wiegel, for example, has tabulated[12] ratios of height and length to their deep water values as a function of relative depth. Using these tables, Hudson's data for each relative depth may be transformed into equivalent data for a single selected reference relative depth. The particular relative depth selected as reference is immaterial, and one is probably as good as another. However, a deep-water-reference relative depth has previously been used[13,14] and has some advantages because deep water values are frequently the actual known values, and the tabulated ratios relate to deep water values.

Accordingly, the author's run-up data for the 1 on 2.5 slope have been referred to a deep water basis ($d/\lambda = 1.0$), and are replotted in Fig. 11. The scales of the graph have been changed to logarithmic (rather than to arithmetic, as the author used) to stretch out the points at the lower steepness values.

TABLE 6.—RUN-UP COMPUTATIONS, 1 ON 2,5 SLOPE

Depth (d) (ft) (1)	Height (H) (ft) (2)	Period (T)(Sec) (3)	Length (λ) (ft) (4)	$\frac{d}{\lambda}$ (5)	$\frac{H}{\lambda}$ (6)	$\frac{R^a}{H}$ (7)	R^a (ft) (8)	$\frac{R^b}{H}$ (9)	R^b (ft) (10)
10	8.00	6.0	101.5	.0986	.0788	.74	5.9	.80	6.4
15	7.56		120.6	.124	.0627	.84	6.4	.84	6.4
18.45	7.40		130.9	.141	.0565	.89	6.6	.86	6.4
25	7.26		145.8	.172	.0497	.93	6.8	.88	6.4
36.9	7.28		163.9	.225	.0444	.96	7.0	.89	6.5
184.5	7.93		184.5	1.00	.0430	.98	7.8	.81	6.4

[a] From Hudson, Fig. 9.
[b] From present discussion, Fig. 3c.

Such stretching seems to indicate a possible tendency for relative run-up to decrease with decreasing steepness below a certain critical steepness value—although this conclusion is largely dependent on the location of but a single point.

All plotted points in this figure now refer to a single relative depth, and a mean curve could be drawn through them. However, the scatter is still large, and a curve somewhat above the mean has been drawn. (This curve is also plotted in Fig. 11 and 13(c), on arithmetic scales.)

Similar curves (also referred to a deep water relative depth, ($d/\lambda = 1.0$) have been drawn for the other slopes for which the author gives data. These are shown as a family of curves in Fig. 12. Also shown in this figure, as a matter of comparison, are similar curves for smooth slopes as derived from previously published reports.[13,14,15,16] The roughness and permeability of the rubble had an obvious reducing effect! These curves are shown in terms of H_o'/T^2 (in which T is the wave period) rather than H_o'/L_o purely as a matter of personnel convenience, because the deep water wave length, L_o, equals $5.12\ T^2$, the two ratios bear a constant relationship, with one being essentially five times the other.

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[12] "Gravity Waves, Tables of Functions," by R. L. Wiegel, Council on Wave Research, Engrg. Foundation, 1954.

[13] "Wave Run-up on Shore Structures," by Thorndike Saville, Jr., Transactions, ASCE, Vol. 123, 1958, p. 139.

[14] "Wave Run-up on Roughened and Permeable Slopes," by R. P. Savage, Transactions, ASCE, Vol. 124, 1959, p. 852.

As these curves are all referred to a deep water relative depth, the designer in using them must compute the deep water wave steepness and height from whatever design wave information he has, and use these values in computing the run-up. Similar curves referred to any other relative depth could, of course, be drawn, and would be equally usable. However, in using them, one must know the relative depth of reference, and be careful to use wave characteristics for that particular reference relative depth. This is true even though this reference depth may not physically exist in the actual field location (as "deep water" does not for many lakes, for example). In this way, one always gets the same run-up computed for a given wave and structure. For the case tabulated previously, and using the curve shown in Fig. 12, this run-up value would be 6.4 ft.

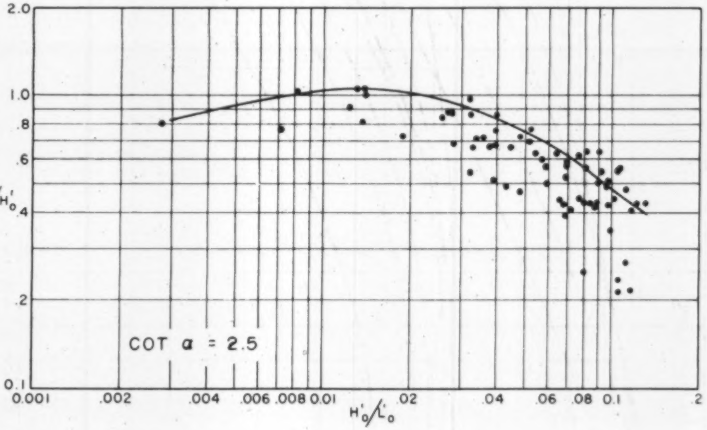

FIG. 11.—WAVE RUN-UP ON RUBBLE SLOPE, 1 ON 2.5 DEEP WATER REFERENCE RELATIVE DEPTH

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[15] "Wave Run-up on Composite Slopes," by Thorndike Saville Jr., Proceedings, 6th Internatl. Conf. on Coastal Engrg., Council on Wave Research, Engrg. Foundation, 1958.

[16] "Shore Protection Planning and Design," Beach Erosion Bd., Tech. Report No. 4, U. S. Army Engr. Beach Erosion Bd., Washington, D. C.

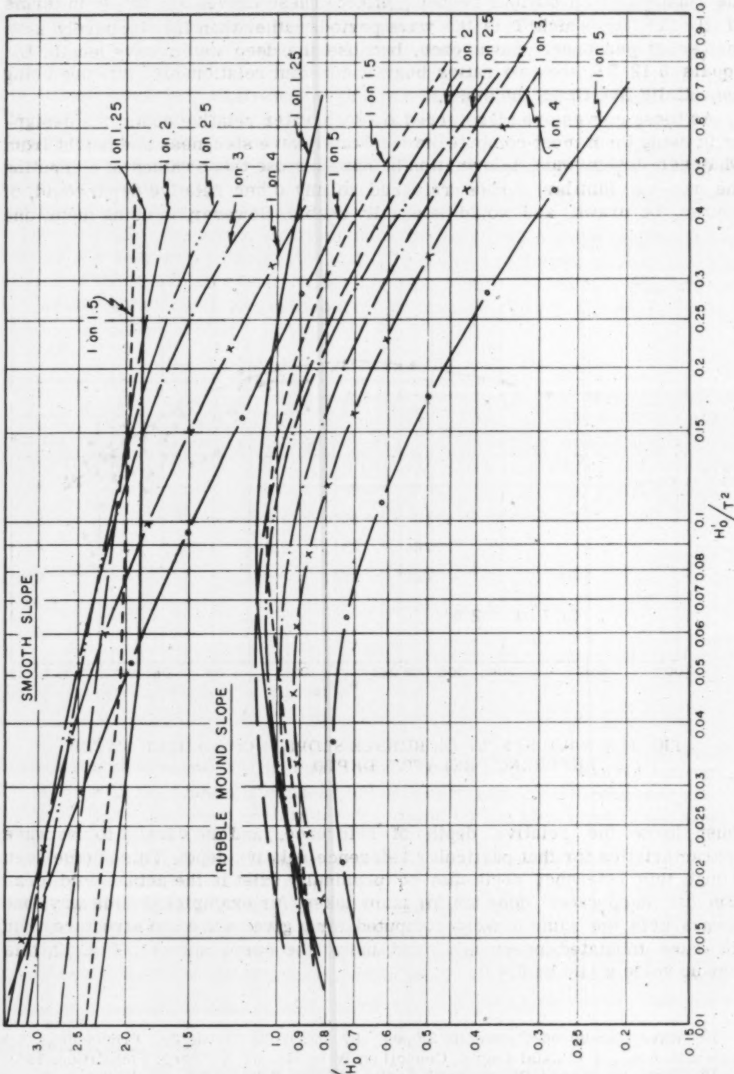

FIG. 12.—WAVE RUN-UP ON RUBBLE MOUND AND SMOOTH SLOPES (FOR VALUES OF $d/H_o > 3$)

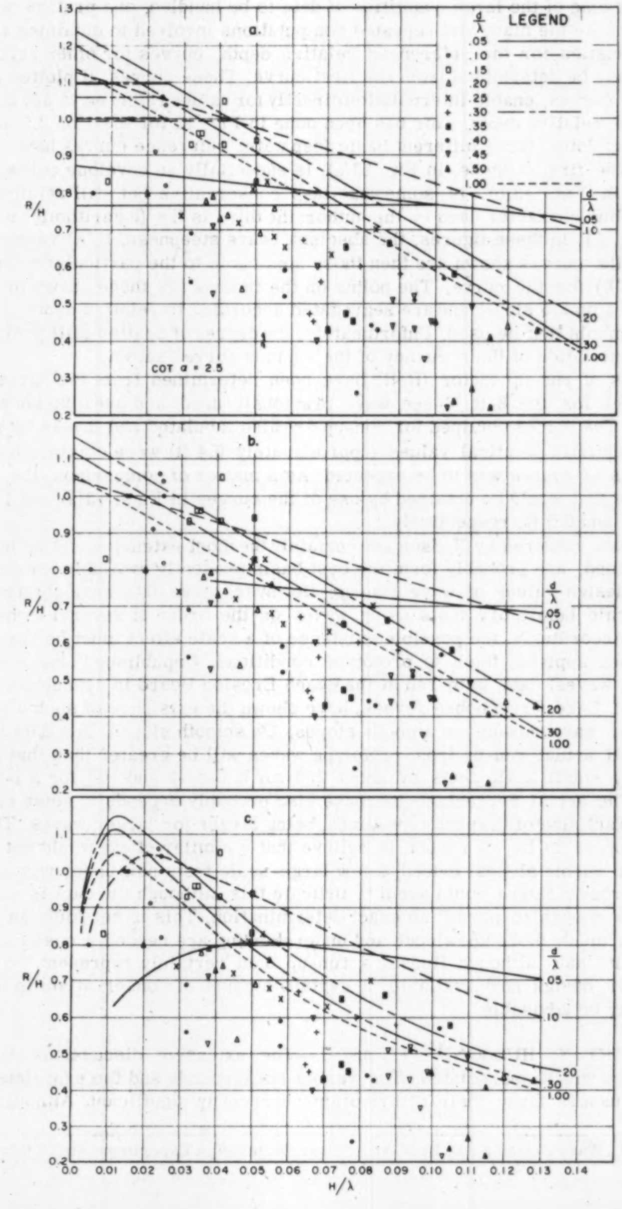

FIG. 13.—WAVE RUN-UP ON RUBBLE SLOPE, 1 ON 2.5. RELATIONSHIP TO RELATIVE DEPTH FOR SEVERAL POSSIBLE RUN-UP CURVES

If, because of the large quantities of data to be handled, one prefers not to have to make the many and repeated computations involved in obtaining wave characteristics for the referenced relative depth, curves for other relative depths may be determined from the first curve. These curves, if plotted as a family of curves, enable interpolation directly for values referred to any other reference relative depth. This has been done in Fig. 13 for the 1 on 2.5 slope data. In so doing, three different basic deep water reference curves have been drawn. The first of these, in Fig. 13(a), is essentially an envelope curve; the second, in Fig. 13(b), is somewhat less conservative but still retains the straight line character used by the author; the third is the (logarithmic) curve from Fig. 11. In these figures, the abscissa (wave steepness, H/λ) values for each of the curves shown are then those applicable to the particular relative depth (d/λ) for the curve. The points on the figures are those shown in Fig. 7 for the 1 on 2.5 slope, and are segregated according to relative depth by the same symbols that he used. Unfortunately, the degree of scatter still prevents a good estimation of the accuracy of the various curves shown.

Values of run-up factor (R/H) have been determined from the curves in Fig. 13(c) for the 8-ft, 6-sec wave previously used, and are also shown in Table 1. The values obtained for run-up are also tabulated and it may be seen that essentially identical values (approximately 6.4 ft) were obtained for all cases, as of course was to be expected. As a matter of comparison, the run-up values that would be obtained by use of the curves in Figs. 13(a) and 13(b), were 7.1 and 6.5 ft, respectively.

The data gathered by Hudson are certainly the most extensive and inclusive yet obtained, and probably form the best basis presently available for determining design values of wave run-up. However, these data were obtained in small scale laboratory tests using waves on the order of several inches in height. Accordingly, the possible existence of a scale effect must be borne in mind when applying them to prototype conditions. Unpublished, large scale (2 ft-5 ft waves) data, gathered at the Beach Erosion Board in connection with design of Lake Okeechobee levees, have shown the existence of such a scale effect for wave run-up on smooth slopes. On smooth slopes, this data indicates that actual run-up from prototype waves will be greater than that predicted by small scale tests by about 20% for a 1-on-3 and 10% for a 1-on-6 slope. The actual percentage increase also probably depends to some extent on the exact size of the prototype waves, being larger for larger waves. There would appear to be no reason to believe that a similar effect would not also exist for rubble slopes. Indeed, a few large-scale tests now under way at the Beach Erosion Board would seem to indicate this, although the data is not yet complete enough to permit an exact determination. This is certainly an area in which much more knowledge and understanding are needed. Consequently, it is felt that, although Hudson's run-up data certainly represent the best basis for design now available, a certain amount of conservatism in using them may be advisable.

ROBERT Y. HUDSON,[17] F. ASCE.—The extensive discussions of this paper are much appreciated. The varied backgrounds and the experience of the discussers make their contributions especially significant. Although the discussion by Carvalho and Vera-Cruz will probably be appreciated more by those engaged in laboratory tests of rubble breakwaters, design engineers in America are also indebted to these gentlemen for the information presented concerning the full-scale breakwater from which Iribarren obtained the data used to compute the coefficient (K′). The discussions by Slichter, Booth, and Lillevang are valued because they represent the ideas of practicing engineers with many years experience in the design and construction of rubble breakwaters. Jones' discussion provides excellent, but provisional, data concerning the stability of riprap cover layers for fill slopes. The stability of cover layers for rubble breakwaters and the stability of riprap cover layers for fill slopes are closely related phenomena, especially from the standpoint of laboratory testing techniques and analyses. However, there is considerable difference in the relative importance of the variables, and in the practical aspects of design.

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[17] Chf., Wave Action Sect., U. S. Army Engr. Waterways Experiment Sta., Vicksburg, Miss.

In each discussion, the thought was expressed that the science of rubble breakwater design is still in the formative stage, and that considerable work remains to be done before stability formulas can be relied on for exact solutions. The writer concurs in this evaluation of the status. Yet, he cannot agree completely with Slichter's comment to the effect that "although the guide lines contributed by the author's work are valuable aids, the design of breakwaters must remain largely in an area dominated by sound judgment based on observations of past experience." It is believed that the observations of past experience should include observations of small-scale tests as well as observations of full-scale structures, and that the formulation of sound judgment should be based both on the relationships between variables, determined by use of small-scale models, and on lessons learned from observation of full-scale breakwaters. It is agreed, however, that the designer cannot afford to use the results of small-scale tests blindly, and that, after all possible model tests have been completed on rubble breakwaters, there will still remain a large area in design in which use of the intuition and practical experience of the designer will be necessary.

Slichter doubts that there is any practical design significance in abandoning the Iribarren formula, in view of the small size of the armor units tested relative to those of a full-scale structure. The reasons for abandoning the Iribarren formula were in no way related to the effects of model scale on the accuracy of test results. It is believed that the scale of the tests is sufficient to insure an adequate degree of dynamic similarity, model to prototype, if it can be assumed that the shape factor of the armor units, the placing of armor units, and the characteristics of incident wave trains are sufficiently similar. It is hoped that the testing programs, now in progress at the Corps of Engineers, U. S. Army Waterways Experiment Station and the Beach Erosion Board laboratory, can determine the effects of these variables.

For those who favor the original Iribarren formula (Eq. 1 of the author's paper), the comparison of results shown in Table 7, obtained by substitution in the formulas of Iribarren and the author, should be informative. In Table 7 $(W_r)_I$ and $(W_r)_H$ are the weights of quarrystone armor units required for stability for the different slopes, assuming equal wave heights, obtained by substituting in the Iribarren and Hudson formulas, respectively. Values of 0.015 for K′ and 3.2 for K_Δ were used. It is noted from this comparison that (a) the formulas agree for a slope of 1-on-2, (b) the weights of quarrystone armor units required for stability, as determined from the Iribarren formula, are approximately one-half those obtained from the author's formula when the slope is 1-on-3 or flatter, (c) the effects of small changes in μ are very large for steep slopes, and (d) the weights of armor rock given by the Iribarren formula for slopes of 1-on-1-1/2 and 1-on-1-1/3 are exceptionally high, compared with the author's formula, especially for low values of μ.

Slichter questions the assumption, made in developing the general functional equation (Eq. 13), that friction between armor units can be neglected. This assumption is, of course, not strictly correct, but it is believed sufficiently accurate for use in developing the functional equation. This belief is based on several years of experience in testing small-scale rubble breakwaters to the point of failure. It was concluded from observing these tests that friction between armor units can be a primary resisting force for pell-mell placed units only when the entire cover layer is on the verge of sliding downslope. It was noticed that, for pell-mell placed armor units, there is considerable failure of individual units, by forces that lift and roll the units out of their nested positions, before failure occurs by sliding of the cover-layer mass. This is not true, however, for the condition in which armor units are carefully selected for shape and are positioned individually to obtain maximum wedging action. For this condition, especially for the steeper slopes, the cover layer usually fails by sliding along the boundary between the cover-layer bottom and the first underlayer. It is hoped that the effects of shape factor and special placing techniques can be determined in future tests.

TABLE 7.—COMPARISON OF THE IRIBARREN AND HUDSON FORMULAS

cot α	$(W_r)\,I/(W_r)\,H$		
	μ = 1.00	μ = 1.05	μ = 1.10
1-1/3	8.0	5.4	3.9
1-1/2	3.4	2.6	2.1
2	1.1	1.0	0.8
3	0.6	0.5	0.5
4	0.5	0.5	0.5
5	0.5	0.5	0.5

Carvalho and Vera-Cruz indicate the following beliefs: (a) the stability of rubble breakwaters situated in relatively shallow water on sandy bottoms is affected by the increased specific gravity of the water resulting from the great amounts of sand stirred up by the waves; (b) the inertia force is important and should be investigated; (c) in the analytical basis of the stability equation, an effort should be made to express the vectorial character of the wave forces; (d) the most adequate criterion for the no-damage condition should be one in which only the active portion of the breakwater, and one or two layers of the protective cover, are taken into account; (e) safety factors should be determined for each type of breakwater section; (f) Iribarren's formula is susceptible of experimental verification, if it is assumed that μ = 1, for all slopes flatter than 1-on-1, and provided that K′ is not considered constant; and (g) K_Δ should not be called a shape coefficient. The writer agrees with the ideas expressed in items (d) and (e). The work explained in the original paper represents the first phase of a comprehensive testing program, and it is agreed that the criterion used for the no-damage tests, and the damage or safety-factor data, are applicable directly only to the type of sections used in the tests. Tests in progress (1960) compare damage to the cover layer with the volume of armor units in the cover layer itself.

The idea expressed in item (a) is new to the writer, and it should no doubt be investigated. It is predicted, however, that tests will show that the stirring-up of sand in the water by wave action does not appreciably affect the stability of rubble breakwaters. The question as to whether the forces of inertia (Eq. 3) are important, concerns a very complicated phenomena, and the writer is not qualified to argue the point. However, the success attained in correlating the test results, using the derived functional equation, in which the effects of inertia forces were included in the experimental coefficient K_Δ, indicates that the assumption made is sufficiently accurate. Also, the work of McNown and Keulegan,[18] concerning the relation of drag and inertia forces on flat plates and cylindrical bodies in periodic motion, appears to indicate that the time available for the formation and shedding of vortexes is not sufficient, in the flow around breakwater armor units caused by short-period wave action, to result in large inertia forces. It is doubted that an attempt to express the vectorial character of the wave forces in the functional equation would be successful (item c above). The direction of the wave forces vary with d/λ, H/λ, H/d, β, σ, cot α, h/H, and t/T. Thus, it was decided to omit the direction variable from the general stability equation, and let the model define the importance and the effects of its function. The success achieved to date in correlating the test results using Eq. 13 appear to substantiate the correctness of this decision.

The writer agrees that Iribarren's formula is susceptible to experimental verification if it is assumed that $\mu = 1$. However, this is tantamount to assuming that friction is important at the time the formula is derived, then assuming it is unimportant for the purpose of experimental verification. It is believed better to let the model tests define the effects of friction. Also, Iribarren's formula includes the term involving the breakwater slope (cos α - sin α), that originated from an assumption with respect to the directions of both wave forces and friction forces. The inadequacy of this assumption is believed to be the primary reason that K′ varies so considerably with breakwater slope.

The information presented by Slichter and Booth concerning the design of breakwaters in Hawaii using tribars and tetrapods should be very useful to other designers. It is good to know that the test results obtained during the model study conducted at the Waterways Experiment Station to determine the optimum design for repairing the Nawiliwili Harbor breakwater, have been partly verified by the prototype breakwater's ability to resist the action of large storm waves.

This breakwater was designed, based on the model test results, to be stable for waves as large as 24 ft in height. The storm referred to by Booth was hurricane "Dot," that passed directly over Kauai, the island on which the Nawiliwili breakwater is situated. Although an accurate estimate of the heights of waves that attacked the breakwater is not presently available, photographs taken before the height of the storm show considerable overtopping of the breakwater. By comparison with similar photographs taken during the model study, it is estimated that the waves were greater than 20 ft in height.

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[18] "Vortex Formation and Resistance in Periodic Motion," by J. S. McNown and G. H. Keulegan, Proceedings, ASCE, Vol. 85, No. EM1, January, 1959.

The writer agrees with Booth's statement that values of the experimentally determined coefficient K_Δ, presented in the author's paper, should not be considered as final. However, the information presented by Booth concerning the Nawiliwili Harbor breakwater and the information presented by Deignan[19] concerning the Crescent City breakwater, indicate that the model test results can be used to predict the action of prototype structures with very good accuracy. The large-scale tests being conducted by the Beach Erosion Board for the Waterways Experiment Station, mentioned by Booth, are not complete at this time, but preliminary results indicate that the author's values of K_Δ for quarrystone are conservative.

The writer agrees with Lillevang that the placing of armor units in such a way as to effect wedging action and interlocking between adjacent units will increase the stability of rubble breakwaters, compared with the stability obtained by pell-mell placement. However, maximum benefit of especially placed armor units cannot be obtained unless all units down to an elevation equal to about -H ft, referred to swl, are so placed. The reason for this is that hydrostatic pressure of water in the cover layer is considerable, and is maximum at an elevation immediately above the trough of the wave on the structure. If the special placing of armor units is carried to the extreme, voids in the cover layer may be decreased sufficiently to cause failure due primarily to the hydrostatic pressure of water trapped in the cover layer. The test data presented in Table 8 answers Lillevang's request for information concerning the effects of shape factor on the stability of cover layers.

TABLE 8.—EFFECTS OF ARMOR UNIT SHAPE FACTOR ON STABILITY OF BREAKWATER STEM IN RELATIVELY DEEP WATER[a]

Armor Unit (1)	n (2)	Placement (3)	K_Δ (4)	k_Δ (5)	P, percentage (6)
Quarrystone	2	Pell-mell	2.6	1.0	40
Tetrapod	2	Pell-mell	8.3	1.0	50
Tribar	2	Pell-mell	12.0	1.0	54
Tribar	1	Regular	25.0	1.1	47
Tetrahedron	2	Pell-mell	3.0	1.0	40
Quadripod	2	Pell-mell	8.3	1.0	50

[a] $\cot \alpha \leq 1.5$

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[19] "Breakwater at Crescent City, California," by John E. Deignan, Proceedings, ASCE, Vol. 85, No. WW3, September, 1959.

These data were obtained using water of considerable depth, relative to wave height, and for the no-overtopping condition. The results of all tests to date are provisional to the extent that the effects of scale, variations in wave height (heights $> H_{1/3}$) in storm-wave trains, placing techniques, overtopping waves, waves breaking directly on the structure, and the angle of wave attack have not been determined. However, when waves do not break directly on the structure, and when there is no overtopping, it is believed that use of the values of K_{Δ} in the Table 8 will result in conservative designs. It is believed, also, that the relative economy of rubble breakwaters, similar except for the shape of armor unit used in the cover layer, can be determined with considerable accuracy, using the writer's formula and the above values of K_{Δ}, k_{Δ}, and P.

The information presented by Jones concerning the measured value of the friction coefficient (μ) is valuable design data. The average value of $\mu = 0.75$, determined for dumped riprap, would increase the required stone weights nearly four times for cot $\alpha = 2$, as explained by Jones, compared with the use of $\mu = 1.0$ and $K' = 0.015$ in Iribarren's formula. It should be realized, however, that the value of $K' = 0.015$ should not be used in Iribarren's formula for riprap cover layers. A new value of K' for riprap would need to be determined, using, in this determination, the new value of $\mu = 0.75$ for the friction factor in Iribarren's formula.

Saville has correctly pointed out that both wave height (H) and wave length (λ), and thus wave steepness (H/λ), depend on depth of water in which H and λ are determined, and that different values of run-up (R) are determined from the author's run-up curves, depending on just where the wave characteristics are measured. However, this fact does not detract from the usefulness or accuracy of the author's run-up curves, as stated by Saville. In the paper, H was defined as the wave height measured at the location of the proposed breakwater. Thus, there should be no confusion as to which value of H/λ to use, because there is only one value of H/λ corresponding to a selected breakwater position and a given deep-water design wave H_o/λ_o. The dimensions of a deep-water design wave are usually selected on the basis of existing hindcasting techniques, and the corresponding value of H/λ, for depth (d) of the selected breakwater location, is determined by constructing wave-refraction diagrams for the area between deep water and the proposed structure location. It is believed that the method of plotting the run-up data selected by the author is not only correct from an analytical standpoint, but also is more convenient and less confusing than the curves proposed by Saville. Also, the tests from which the run-up data were obtained were conducted using water of considerable depth relative to wave height (H/d varied from a minimum of 0.28 to a maximum of 0.35).

Thus, it is doubtful whether the data can be used for the condition of breaking waves, as did Saville in his example (an 8-ft wave of 6-sec period will break in a water depth of about 10.5 ft). It is hoped that future tests can be conducted for the condition of gradually increasing depth seaward of the breakwater location, using both breaking and nonbreaking waves, and a larger range of values of d/λ. We are indebted to Saville for pointing out the possible effects of scale on wave run-up. The writer was not aware that scale effects for run-up on rubble breakwaters were appreciable. Until more data are available concerning scale effects on wave run-up, the crowns of breakwaters probably should be designed to withstand some over-topping, without failure, whenever the run-up curves shown in the paper are used.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3214

DESIGN OF SEA-WALLS AND BREAKWATERS

By Ira A. Hunt, Jr.[1] M. ASCE

SYNOPSIS

Sea-wall and breakwater design from the standpoint of the manner and effectiveness with which they reflect or dissipate the incident wave energy are presented herein. Various laboratory results are considered and their implications reconciled, particularly as regard wave up-rush.

FOREWORD

The building of protective sea-walls and breakwaters along coastal and inland shores has become imperative in many places because of our rapidly increasing population and expanding industries. Economics has dictated that these structures cannot be overdesigned, yet the impelling need for protecting life and property makes it essential that they be safely designed. Destructive forces of water masses have been cause for concern to many countries for centuries. The attention of the world was focused on the Netherlands and England when the storm flood of February 1, 1953 breached their dikes and sea-walls causing great loss of life and damage to property. The writer had the invaluable experience of assessing firsthand the causes of the dike failures, of which a detailed report was submitted to the Chief of Engineers, U. S. Army.[2] The sight of fertile lands and deserted villages inundated as far as the eye can see makes a lasting impression of the vital need for proper forecasting techniques and design of sea defenses. Fig. 1 shows waves attacking buildings in inundated polder in the town of Kerkwerve, on the island of Schouwen, the Netherlands. Although some of the sea-walls failed because of mechanical defects and im-

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[1] Major, Corps of Engrs., U. S. Lake Survey, Detroit, Mich.

[2] "The Storm Flood on February 1, 1953 and Its Effect on the Netherlands," by I. A. Hunt, (unpublished) Corps of Engrs., Washington 25, D. C., December, 1953.

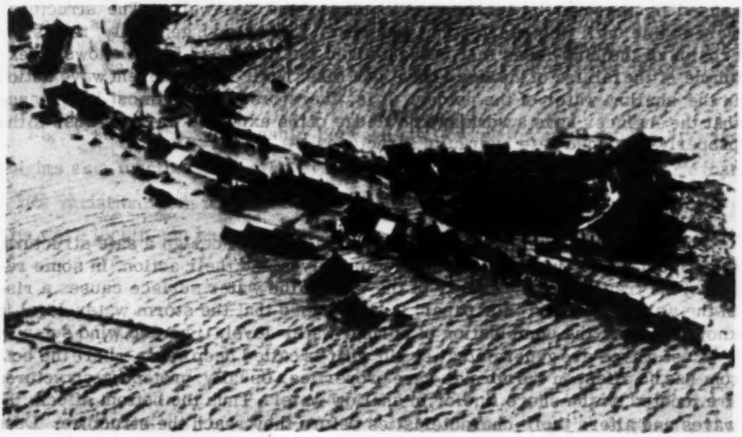

FIG. 1.—WAVES ATTACKING BUILDINGS IN INUNDATED POLDER

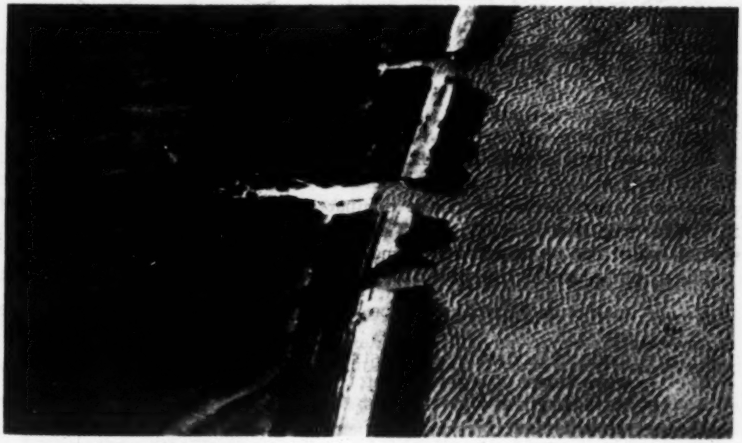

FIG. 2.—FAILURE OF THE INSIDE FACE OF A DIKE

proper drainage, it is reliable knowledge that the majority of all failures were caused by the overflowing or overtopping of the sea-walls. The structures were stable until this overtopping occurred (Fig. 2) and, generally, it was the erosion of the land-faces that brought about the failures. Fig. 2 shows an example of the failure of the inside face of a dike. Notice the different wave action in the shallow water of the polder. It is, therefore, of the utmost importance that the engineer have a working knowledge of the expected wave up-rush to the properly-designed safe hydraulic structures.

WAVE CHARACTERISTICS

Because waves are the prime destructive force, to design a safe structure, facts must be obtained about the expected waves and their action. In some regions, the transfer of the energy of the wind to the water surface causes a rise in the water-level. In this paper, it is assumed that the storm water-level is known, that is, the combination of normal water-level, tide, and wind set-up.

Most waves are generated at sea in what is called deep water where the bottom has no affect on the normal wave properties. Usually, protective structures are located on the shore line or in shallow water. Thus the bottom affects the waves and alters their characteristics before they reach the structure. Deep water wave characteristics can be computed using the fetch-type formula of H. V. Sverdrup and W. H. Munk.[3] The characteristics of waves generated in shallow water can be obtained by using relationships given elsewhere.[4]

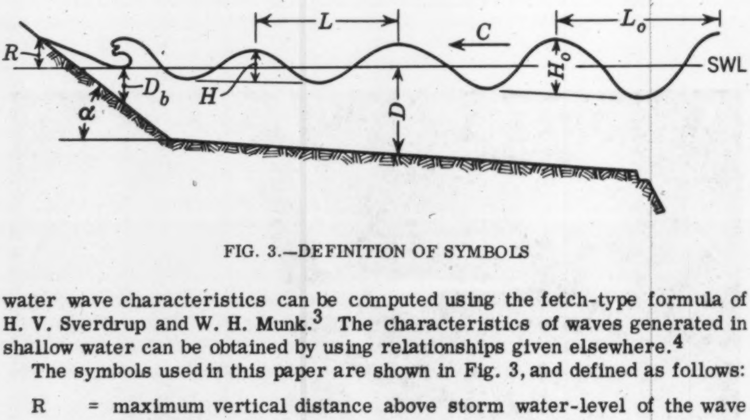

FIG. 3.—DEFINITION OF SYMBOLS

The symbols used in this paper are shown in Fig. 3, and defined as follows:

R = maximum vertical distance above storm water-level of the wave up-rush;
α = angle of inclination of the structure with the horizontal;
H = wave height at a given point;
L = wave length at a given point;
T = wave period;
C = wave velocity;
D = water depth from bottom to the storm water-level; and
SWL = storm water-level.

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[3] "Wind, Sea, and Swell: Theory of Relations for Forecasting," by H. V. Sverdrup and W. H. Munk, U. S. Hydrographic Office, Pub. No. 601, March, 1947.

[4] "Waves and Wind Tides in Shallow Lakes and Reservoirs," Dist. Engr., Corps of Engrs., Jacksonville, Fla., June, 1955.

The subscripts o refer to deep water wave conditions. The subscripts b refer to the wave condition at the point of breaking.

There are several relationships[5] which will be presented briefly to assist in clarifying the change in wave properties as a wave is propagated from deep to shallow water. The term "deep water" usually connotes water deeper than one-half the surface wave length, $\frac{D}{L} > \frac{1}{2}$.

Fig. 4 shows the relationship between shallow water and deep water wave heights as a function of $\frac{D}{L_o}$. For values of $\frac{D}{L_o} > .033$ there is never more than a 10% variation from unity in the value of $\frac{H}{H_o}$.

The wave length can be computed as

$$\frac{L}{L_o} = \tanh \frac{2 \pi D}{L} \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

Munk[6] has shown that

$$\frac{H_b}{D_b} = 0.78 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

M. F. Suquet[7] has computed the relationship between $\frac{D_b}{H_o}$ and $\frac{H_o}{L_o}$. This is shown in Fig. 5.

With these relationships and the general formula for the velocity of a wave

$$C^2 = \frac{g L}{2 \pi} \tanh \frac{2 \pi D}{L} \quad \ldots\ldots\ldots\ldots \quad (3)$$

it is possible to compute most of the necessary wave characteristics. It should be remembered that the wave period, T, may be considered constant as the wave approaches the shore.

It will be presumed that the waves approach the structure frontally and the wave heights are unaffected by refraction. However, if the structure or coastline is fronted by a gentle sloping beach, obliquely approaching waves will be bent so that the wave crests will aline themselves with the bottom contours and the waves will advance almost frontally.

The velocity at which the energy of a wave is propagated shoreward differs from the wave velocity. In deep water, the wave energy moves with one-half the wave speed, whereas in shallow water, that is, when $\frac{D}{L} < .04$, the energy is propagated with the speed of the waves. The total energy of a shallow water wave is slightly less than that of a deep water wave. The energy of a shallow water wave is concentrated closer to the water surface than it is for a deep water wave. D. D. Gaillard[8] has shown that for a wave $\frac{H}{L} = \frac{1}{15}$ and $\frac{D}{L} = \frac{1}{10}$ the total

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[5] "Shore Protection Planning and Design," Beach Erosion Bd., Corps of Engrs., Tech. Report No. 4, June, 1954.

[6] "The Solitary Wave Theory and Its Application to Surf Problems," by W. H. Munk, Annals, N. Y. Academy of Science, Vol. 51, May, 1949.

[7] "Study of Breaking Waves," by M. F. Suquet, La Houille Blanche, No. 3, Grenoble, France, 1950.

[8] "Wave Action Relation to Engineering Structures," by D. D. Gaillard, Corps of Engrs., Professional Paper No. 31, 1904.

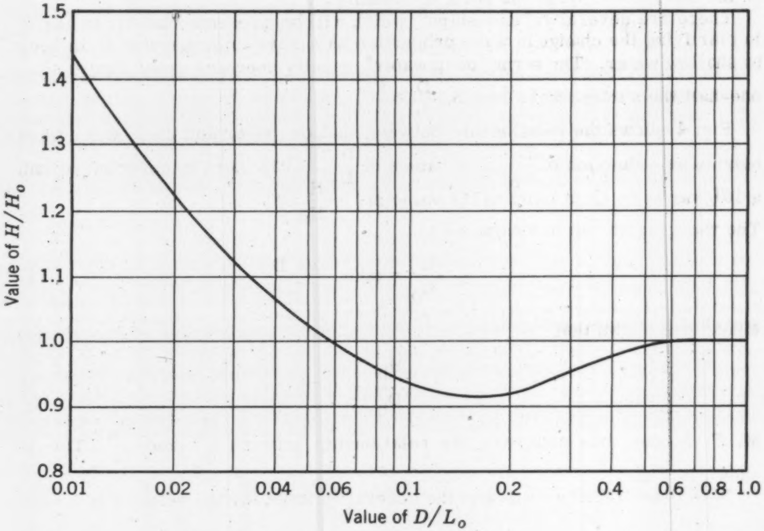

FIG. 4.—RELATION OF SHALLOW WATER AND DEEP WATER WAVE HEIGHTS AS A FUNCTION OF D/L_o

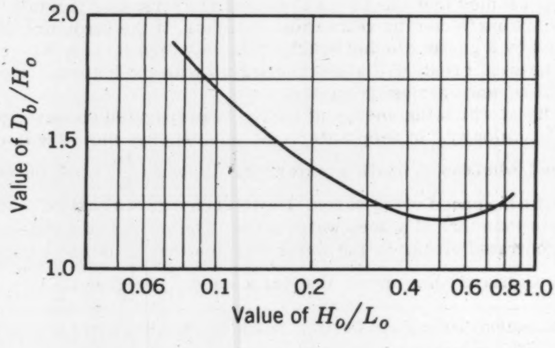

FIG. 5.—RELATIONSHIP OF RATIO OF DEPTH OF BREAKING TO DEEP WATER WAVE HEIGHT, WITH DEEP WATER WAVE STEEPNESS

theoretical energy of the shallow water wave is 95% of that of a deep water wave of equal height and length. Nevertheless, in the case of the shallow water wave, an amount of energy equal to more than half the total energy of the deep water lies above the horizontal plane situated at a distance of .03 L below the line of surface orbit centers, while in the case of the deep water wave approximately 31% of the total energy of the wave lies above the same plane.

Thus it can be seen that the wave characteristics and the depth of the water at the approach of a structure have a great part to play in the forward transfer of energy and the concentration of energy in the wave. In 1953 the writer summarized all of the general formulas known to him which were being utilized to compute wave up-rush.[2] One fact was outstanding; wave characteristics, except for the approaching wave heights, did not enter into these formulas. The importance of wave characteristics and demonstrations of how they affect the design of sea-walls and breakwaters are emphasized.

ENERGY DISSIPATION

The key to proper design is an insight into the manner in which the oncoming wave energy is dissipated. In general terms, the energy of a wave striking a sea-wall or shore-line is dissipated by:

1. Reflection.
2. An increase in the potential energy, that is, wave up-rush.
3. Heat.
    a. Generated by the turbulence of the breaking of the wave.
    b. Generated by the roughness of the structure.
    c. Generated by the mixing in the voids of a permeable structure.

In the case of a rubblemound breakwater with a high percentage of voids, the energy may be transferred directly from the ocean side to the harbor side.

REFLECTION

Reflection is studied first because it is, in the writer's opinion, the most important single factor to be considered in the design of a sea-wall or breakwater. When an oncoming wave strikes a structure or even the natural coast, it can be partially or totally reflected. In a port, the local agitation of the sea is increased with reflection, and it can become a disturbing factor. Accounts show that breakwaters have collapsed or have been damaged because of erosion in front of the structure caused by the reflection of waves. Measures, such as the construction of a rubblemound in front of structures, have been effected to reduce wave reflection and thereby increase the stability of the bottom. If it is at all possible, the reflection should be kept to a minimum.

Excellent papers presenting reflection have been written by M. Miche[9, 10] and Iribarren and Nogales.[11] Miche has determined the maximum value of the

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[9] "Undulatory Movement of the Sea," by M. Miche, Annals des Ponts et Chaussées, Paris, 1944.

[10] "The Reflecting Power of Maritime Works Exposed to Action of the Swell," by M. Miche, Annals des Ponts et Chaussées, Paris, May-June, 1951.

[11] "Protection of Ports," by Iribarren and Nogales, Seventeenth Congress, Internatl. Assoc. of Navigation Congress, Ocean Navg., Sect. 2, Communication 4, Lisbon, Portugal, 1947.

wave steepness at sea, $\left(\frac{H_o}{L_o}\right)_{max}$ or $\gamma_{o\,max}$, of a wave which is theoretically capable of being totally reflected by an indefinite inclined plane making the angle α with the horizontal as

$$\gamma_{o\,max} = \sqrt{\frac{2\,\alpha}{\pi}} \quad \frac{\sin^2\alpha}{\pi} \quad \ldots\ldots\ldots\ldots\ldots \quad (4)$$

If the profile of the wave is considered, the maximum steepness of the wave $\frac{dy}{dx}$ is equal to tan α. For a greater steepness there will be breaking of the incident wave or the dissipation of power preventing total reflection. Miche defines the reflecting power, R, of a structure or of a coast as the ratio of the amplitude H_1 of the reflected swell to the amplitude H of the incident swell

$$R = \frac{H_1}{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

If γ_o is the wave steepness at sea and $\gamma_{o\,max}$ is the steepness compatible with total reflection, the part theoretically reflected, R', has the value

$$R' = \frac{\gamma_{o\,max}}{\gamma_o} \quad R' < 1 \quad \ldots\ldots\ldots\ldots \quad (6)$$

According to results, Miche states that it should be expected that there is a certain diminution in volume of the reflected swell due to the turbulence produced in the orbital movement of the wave in contact with the slope. Thus, there is reason to introduce a coefficient ρ for intrinsic reflection of the slope, apparently independent of the slope and which should have a value of 0.8 or 0.9 for smooth slopes. The effectively reflected part, that is, the reflecting power R is, therefore, equal to

$$R = \frac{H_1}{H} = \rho\,R' = \rho\;\frac{\gamma_{o\,max}}{\gamma_o} \ldots\ldots\ldots\ldots \quad (7)$$

Iribarren and Nogales, utilizing different assumptions, have defined a critical slope as such that a reduction in the slope will produce a breaking of the wave, and an increase in the slope will cause a surging action and reflection. The value which they obtained for the critical slope is

$$\tan\alpha = i = \frac{8}{T}\;\sqrt{\frac{H}{2\,g}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

For a given wave, a slope with the preceding value will give a reflecting power of one-half

$$R = \frac{H_1}{H} = \frac{1}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

If it is assumed that $H \approx H_0$, which is a logical assumption for most actual shore conditions (Fig. 4), then Eq. 8 becomes

$$\gamma_0 = \frac{i^2}{5.1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

Miche states that his formula is less accurate for slight slopes than for the steeper slopes. Considering values of $\alpha \geq 11°$, that is, $\tan \alpha = i \geq \frac{1}{5}$, the value of $\gamma_{o\,max}$ can be approximated from Eq. 4 by

$$\gamma_{o\,max} = \frac{i^2}{8.1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

Now $\frac{H_1}{H} = \rho \frac{\gamma_{o\,max}}{\gamma_o}$, but the γ_0 of Iribarren and Nogales is for the condition $\frac{H_1}{H} = \frac{1}{2}$.

Therefore, substituting the values of Eq. 10 and 11,

$$\frac{H_1}{H} = \frac{1}{2} = \rho \frac{\frac{i^2}{8.1}}{\frac{i^2}{5.1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

and

$$\rho = \frac{8.1}{2\,(5.1)} = .8$$

And, since $\rho = .8$ in Eq. 12, the results of Iribarren and Miche are comparable. For an impermeable smooth slope, Miche has stated that the value of ρ should be approximately 0.8. The formula of Iribarren was derived for a smooth impermeable slope.

It was previously stated that sea-walls and breakwaters should be constructed to keep the reflection of incident waves to a minimum. It has been shown theoretically that if $\gamma_0 > \frac{i^2}{5.1}$, breaking will occur and the reflecting power will be less than one-half. Therefore, it is of importance to determine if this criteria of Iribarren and Nogales holds true in actual cases. They conducted several experiments which are summarized in Table 1. The formula of Iribarren and Nogales gives results which are close to the average value between total breaking and total reflection, and that the formula of Miche gives excellent results for the slope necessary for total reflection.

J. W. Caldwell states[12] that

> It was noticed that as the test slope is steepened, at some point a troublesome amount of energy would be reflected from the slope in the form of a reflected wave. An investigation of this action showed that for a 6-ft, 7-second wave (prototype) a 1:3 slope destroyed sufficient energy so that

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[12] "The Design of Wave Channels," by J. W. Caldwell, Proceedings, First Conf. on Ships and Waves, October, 1954.

> no reflection problem was in evidence. It was further found that for a 6-ft, 14-second wave a slope of 1:5 could be tested without excessive reflection.

If the formula of Iribarren, for given wave characteristics, will give the slope which will not give excessive reflections, then the answers given by his formula should be comparable to those previously quoted. Table 2 shows that the answers are comparable.[12]

K. N. Granthem performed[13] some of the first tests on wave up-rush in the United States. In his laboratory tests, he noted whether the wave broke on the structure or whether the wave surged up the structure surface without breaking. He depicted four graphs[14] of $\frac{R}{H}$ versus the slope of structure for constant values of $\frac{D}{L}$ and $\frac{H}{L}$. The value of slope given by $i = \sqrt{5.12\,\gamma_o}$ definitely appears to be the limiting slope between the breaking and surging of waves on the structure. The data in Table 3 was derived from Granthem's paper. This table indicates that Iribarren's formula computes the limiting slope remarkably well.

TABLE 1.—LABORATORY DATA OF IRIBARREN AND NOGALES[a]

Wave characteristics		Measurement of slopes $i = \tan\alpha$			Computed Slopes (Iribarren) $i = \frac{8}{T}\sqrt{\frac{H}{2g}}$	Computed Slopes (Miche) $\gamma_{o\,max} = \sqrt{\frac{2\alpha}{\pi}}\,\frac{\sin^2\alpha}{\pi}$
H, in cm	T, in sec	Total Breaking	Total Reflection	Average		
5.5	0.66	0.42	0.86	0.64	0.66	0.80
4.5	0.92	0.29	0.59	0.44	0.42	0.50
4.5	1.00	0.33	0.49	0.41	0.38	0.46

[a] "The Reflecting Power of Maritime Works Exposed to Action of the Swell," by M. Miche, Annals des Ponts et Chausées, Paris, May-June, 1951, Table 1.

The Beach Erosion Board is conducting a systematic investigation of wave up-rush. The writer applied the criterion of Iribarren to dozens of tests that they made in their wave channels and found conclusive evidence that $i = \sqrt{\frac{H}{T^2}}$ is approximately the critical slope for a smooth impermeable structure, that is, for a given wave, a decrease in the slope will definitely cause the wave to break on the structure.

There is excellent evidence to indicate that the value of the slope given by the formula of Iribarren and Nogales

$$i = \frac{8}{T}\sqrt{\frac{H}{2g}} \approx \sqrt{\frac{H}{T^2}} \quad \ldots\ldots\ldots\ldots \quad (13)$$

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[13] "Wave Run-up on Sloping Structures," by K. N. Granthem, Transactions, Amer. Geophysical Union, Vol. 34, No. 5, October, 1953.

[14] Ibid., Fig. 3.

is indeed the limiting slope between breaking and surging waves. In design, therefore, it is most desirable to have the slope of the structure, i, such that it is less than $\sqrt{\frac{H}{T^2}}$ for the design waves. In this way, one is assured of reduced reflections and increased dissipation of energy by the heat generated by the turbulence of the breaking wave.

WAVE UP-RUSH

Another method by which oncoming wave energy is dissipated is by wave up-rush. When a wave breaks and rushes up a beach or structure, some energy is dissipated by the turbulence of the breaking of the wave and the remaining kiretic energy of the wave is transformed into potential energy as it runs up the slope. In this section, only an impermeable smooth structure with a continuous slope will be considered. Once the wave up-rush on this simplified type of structure has been analyzed, an attempt will be made to predict what will happen with rough, porous, and discontinuous sloping structures.

TABLE 2.—LABORATORY DATA OF CALDWELL

Wave characteristics		Slopes determined in the laboratory	Slopes calculated by $i = \sqrt{\frac{H}{T^2}}$
H, in ft	T, in sec		
6	7	.333	.349
6	14	.200	.175

TABLE 3.—LABORATORY DATA OF GRANTHEM

Wave characteristics		Longest measured slope where breaking occurred	Slope computed by Iribarren's formula	Smallest Measured slope where surging occurred
D/L	H/L			
0.066	0.012	. .	21.5°	15°
0.148	0.035	25°	26.5°	30°
0.218	0.071	30°	33.0°	36°
0.434	0.112	35°	37.0°	45°

In formulating the physical law which governs a given natural phenomena, it is normal engineering procedure to form dimensionless parameters from the variables which influence the action studied. The dimensionless parameters evolved can then be studied in the laboratory and a solution effected.

In an investigation to determine the amount of up-rush on a smooth impermeable continuous slope, the following variables should be considered:

R = wave up-rush;
H = wave height, it is assumed $H \approx H_o$;
L = wave length;
D = depth of water;
C = wave velocity, equals $\frac{L}{T}$;
i = slope of the structure, equals $\tan \alpha$;
E = wave energy;
ρ = density of fluid; and
μ = viscosity of the fluid.

Utilizing the Buckingham Pi theorem and the aforementioned variables, the following dimensionless parameters are obtained.

$$\frac{R}{H}, \frac{H}{L}, i, \frac{H}{D}, \frac{H^2 C^2 \rho}{E}, \frac{H C \rho}{\mu}$$

The term $\frac{H C \rho}{\mu}$, which is in the form of the Reynolds number, may be neglected because the viscous effects should be small except for flat beach slopes, which present no engineering problem.

The term $\frac{H^2 C^2 \rho}{E}$ reduces to approximately the value of $\tanh \frac{2 \pi D}{L}$; consequently,

$$\frac{R}{H} = f\left(\frac{H}{L}, i, \frac{H}{D}, \tanh \frac{2 \pi D}{L}\right) \quad \ldots\ldots\ldots\ldots (14)$$

At this point, study in the hydraulics laboratory of the effect which these parameters have on the wave up-rush is required. This can be done easily by varying one parameter at a time while the others remain constant.

Breaking Waves.—The mechanics of breaking waves and surging waves are entirely different. It has previously been determined that whenever the slope of a structure is less than $\left(\frac{H}{T^2}\right)^{1/2}$, waves will break on the structure. In any laboratory investigation, it is of paramount importance to differentiate between waves which break on a structure or beach and waves which surge up the structure or beach. It is also important to note whether oncoming waves break on the structure or on the beach in front of the structure. The up-rush of waves which break on the beach in front of a structure must be considered and computed as breaking on a composite slope and not a continuous slope. Composite slopes will be studied subsequently.

From the studies of the Waterways Experiment Station,[15] it can be shown rather conclusively that the parameter $\frac{H}{D}$ has little, if any, effect on wave run-up as long as the waves break on the structure (Fig. 6). This has also been evidenced by the tests conducted at the Beach Erosion Board.[16] It should definitely not be concluded that the depth, D, has no effect on the run-up because it plays a vital role insofar as it affects the wave characteristics. It is only for waves breaking on a structure that the dimensionless parameter $\frac{H}{D}$ is not of importance.

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[15] "Wave Run-up and Overtopping, Levee Sections, Lake Okeechobee, Florida," Corps of Engrs., Waterways Experiment Sta., Tech. Report No. 2-449, January, 1957.

[16] "Wave Run-up on Shore Structures," by Thorndike Saville, Jr., Proceedings, ASCE, Vol. 82, No. WW 2, April, 1956.

The same series of tests by the Waterways Experiment Station and the Beach Erosion Board measured the up-rush on impermeable structures with varying slopes for given wave conditions. The results indicated that the up-rush is definitely proportional to the slope. This is also illustrated in Fig. 6, therefore,

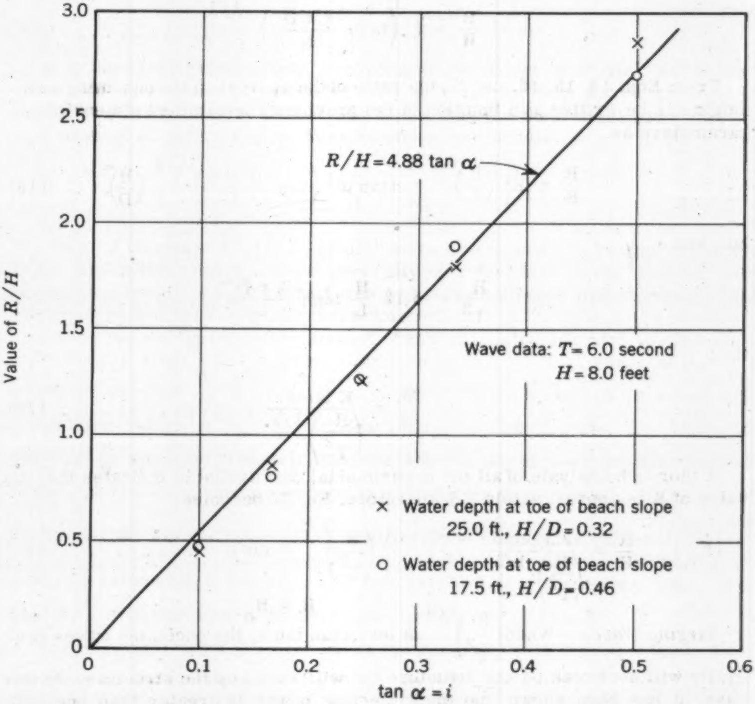

FIG. 6.—RATIO OF WAVE UP-RUSH TO WAVE HEIGHT FOR VARYING BEACH SLOPES AND WATER DEPTHS

$$\frac{R}{H} = C \tan \alpha \qquad (15)$$

For given slopes, the values of C in Eq. 15 were found to be a function of $\left(\frac{H}{L}\right)^{-1/2}$ In which case

$$\frac{R}{H} = f \left(\frac{H}{L}\right)^{-1/2} \quad \ldots\ldots\ldots\ldots\ldots \quad (16)$$

On the other hand, for a constant slope and wave steepness, the data of the Waterways Experiment Station, the Beach Erosion Board, and Granthem indicate that, roughly

$$\frac{R}{H} = f \left(\tanh \frac{2\pi D}{L}\right)^{-1/2} \quad \ldots\ldots\ldots\ldots \quad (17)$$

From Eqs. 14, 15, 16, and 17, the ratio of the up-rush to the oncoming wave-height can be written as a function of the previously determined dimensionless parameters as

$$\frac{R}{H} = K_1^1 \left(\frac{H}{L}\right)^{-1/2} (\tan \alpha)^1 \left(\tanh \frac{2\pi D}{L}\right)^{-1/2} \left(\frac{H}{D}\right)^{\circ} \ldots \quad (18)$$

but, since

$$\frac{H}{T^2} = 5.12 \; \frac{H}{L} \tanh \frac{2\pi D}{L} \quad \ldots\ldots\ldots\ldots \quad (19)$$

then

$$\frac{R}{H} = \frac{K \tan \alpha}{\left(\frac{H}{T^2}\right)^{1/2}} \quad \ldots\ldots\ldots\ldots\ldots \quad (20)$$

A thorough analysis of all the experimental data available indicates that the value of K is approximately 2.3, therefore, Eq. 20 becomes

$$\frac{R}{H} = \frac{2.3 \tan \alpha}{\left(\frac{H}{T^2}\right)^{1/2}} \qquad \left\{ \begin{array}{l} \left(\frac{H}{T^2}\right)^{1/2} > \tan \alpha \\ H \approx H_o \end{array} \right. \quad \ldots\ldots\ldots\ldots \quad (21)$$

Surging Waves.—When $\left(\frac{H}{T^2}\right)^{1/2}$ is less than $\tan \alpha$, the oncoming waves generally will not break on the structure but will surge up the structure. In this case, it has been shown that the reflecting power is greater than one-half. Large quantities of energy are no longer dissipated by breaking, but additional energy is dissipated by reflection. The amount of energy converted into run-up is not immediately decreased, and tests show that the run-up generally increases from 2.3 times the wave-height given by Eq. 21 when $\left(\frac{H}{T^2}\right)^{1/2} = \tan \alpha$

to approximately three times the wave-height at sea

$$\frac{R}{H} \approx 3.0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (22)$$

It may be assumed theoretically that, when the approaching wave-steepness is equal to $\gamma_{o\ max}$, 100% reflection will occur. In this case, since there is no breaking of the wave, the up-rush on a vertical structure can be determined by the formula of M. Sainflou[17]

$$\frac{R}{H} = 1 + \frac{H}{L} \coth \frac{2\pi D}{L} \quad \ldots\ldots\ldots\ldots \quad (23)$$

It is interesting to note that, according to the theory of Miche,[9] an inclination of 45° is capable of reflecting 100% of most incident waves since γ_o is rarely greater than 0.11. Miche has shown that the theoretical wave up-rush on a sloping structure can be determined by the formula

$$\frac{R}{H} = \sqrt{\frac{\pi}{2\alpha}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

The data of Granthem[13] and O. Sibul[18] show this formula to be approximately true. For surging waves, there generally is a decrease in the up-rush with decreasing values of γ_o for structures with slopes steeper than one-half. This can be seen in Fig. 7, wherein experimental data of Granthem are depicted.

As the slopes decrease, the amount of energy dissipated by friction becomes increasingly great. For a given slope, there is a critical range of γ_o 's in which the value of $\frac{R}{H}$ is a maximum, after which the ratio will decrease with decreasing values of γ_o. In the Beach Erosion Board tests, the maximum value of $\frac{R}{H}$ was approximately five, and this was for wave steepnesses that had slight practical applications except in considering tidal waves. For slopes of tan α less than one-half, the ratio $\frac{R}{H}$ will increase slightly for values of $\left(\frac{H}{T^2}\right)^{1/2}$ a little less than tan α; the ratio will then level off at its maximum value; and, finally, $\frac{R}{H}$ will decrease with decreasing values of γ_o (Fig. 8).

The characteristics of storm waves are generally such that $\gamma_o > 0.05$. Thus for steep slopes of α equal to or greater than 45°, the ratio of $\frac{R}{H}$ may be conservatively considered to be approximately 3 in design. Whereas, in design for slopes of α less than 45° it is expedient to utilize Eq. 21 to compute the ratio $\frac{R}{H}$.

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[17] "Treatise on Vertical Breakwaters," by M. Sainflou, Annals des Ponts et Chaussées, Paris, 1928.

[18] "Flow Over Reefs and Structures by Wave Action," by O. Sibul, Wave Research Lab., Univ. of California, Ser. 3, No. 361, June, 1954.

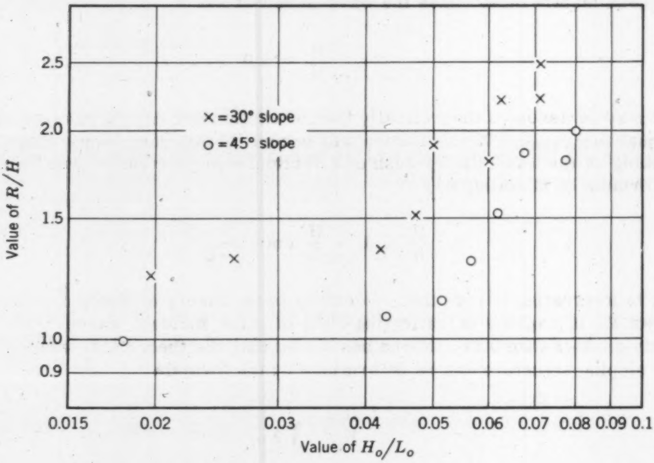

FIG. 7.—RATIO OF WAVE UP-RUSH TO WAVE HEIGHTS FOR SURGING WAVES AND VARYING VALUES OF $\frac{H_O}{L_O}$

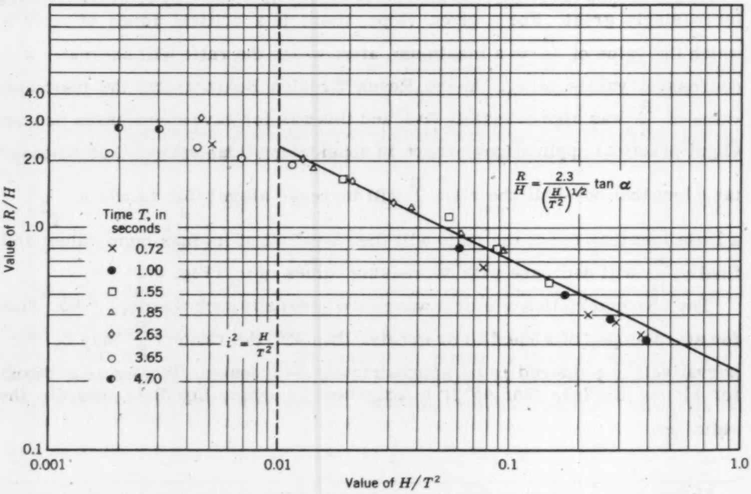

FIG. 8.—RATIO OF WAVE UP-RUSH TO WAVE HEIGHTS FOR CONTINUOUS SMOOTH IMPERMEABLE SLOPE OF 1:10

Composite Slopes and Berms.—It is seldom possible to design protective works with continuous slopes. More often than not, it is advisable, for engineering and economic reasons, to utilize composite slopes or berms (Fig. 9). New construction in England has made use of the berm; excellent examples are the Pitt Level and Dymchurch sea-walls. However, the Dutch were the first to make use of a berm on the seaward side of a sea-wall and they have successfully continued to do so for years. In the Netherlands after the 1953 storm flood, two sections of a dike were observed, one with a wide berm and the other with a narrower berm, which had been exposed to the same wave attack. The section with the wide berm stood intact, whereas the section with the narrower berm was destroyed even though it had a higher crown. There is no doubt that from a hydraulic standpoint, a berm has advantages and it is the purpose of this portion of the paper to analyze up-rush on smooth impermeable structures with berms.

Composite Slopes.—A composite structure is a simplified case of a berm where the wave up-rush is dissipated on the second slope and, as a consequence, a third slope is not necessary. Since a breakwater or a sea-wall is required to resist wave attack under all conditions, it is desirable to design for storm conditions. The ultimate design height is dependent on the degree of protection necessary and the monies available. The slope i_1 should be so designed that all oncoming waves will break on it, that, $H/_T2 > i_1^2$. To insure the waves striking the slope i_1, the break in slope of a composite slope structure should be at the highest expected water level. If the slope i_2 is less than i_1, a reduction in wave up-rush will occur. The curve $H/_T2$ versus R/H for the composite slope will fall in-between and be parallel to the run-up curves

$$\frac{R}{H} = \frac{2.3 \ i_1}{\left(\frac{H}{T^2}\right)^{1/2}} \qquad (25)$$

and

$$\frac{R}{H} = \frac{2.3 \ i_2}{\left(\frac{H}{T^2}\right)^{1/2}} \qquad (26)$$

Analysis of the tests of the Waterways Experiment Station and the Beach Erosion Board clearly indicates that the foregoing statement is true (Fig. 10).

Whenever the water level is well below the break in slope, the wave up-rush can be computed by Eq. 25. If the storm water level should be much above the break in slope so that the waves break on the upper slope, the wave up-rush can be computed by Eq. 26. For proper design, the break in slope in composite structures and the berm height in berm-type structures should be at the highest expected storm water level. This design will give the maximum dissipation of energy with the minimum construction costs. P. Bruun has stated,[19] "Experiments with varying berm height proved that the berm, especially when roughened, gave the least reflection when it was placed at the same height as or a little higher than the highest water level." Dutch engineers also use this design criterion.

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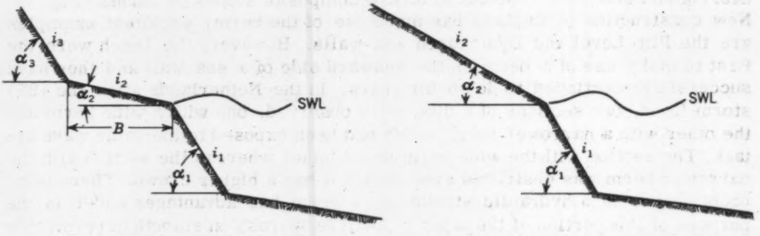
[19] "Breakwaters for Coastal Protection, Hydraulic Principles in Design," by P. Bruun, Eighteenth Navg. Congress, Sect. 2, Question 1, Rome, 1953.


i_3
α_3
i_2
α_2
B
SWL
i_1
α_1
i_2
α_2
SWL
i_1
α_1

(a) BERM-TYPE STRUCTURE (b) COMPOSITE SLOPE STRUCTURE

FIG. 9.—BERM AND COMPOSITE SLOPE STRUCTURES

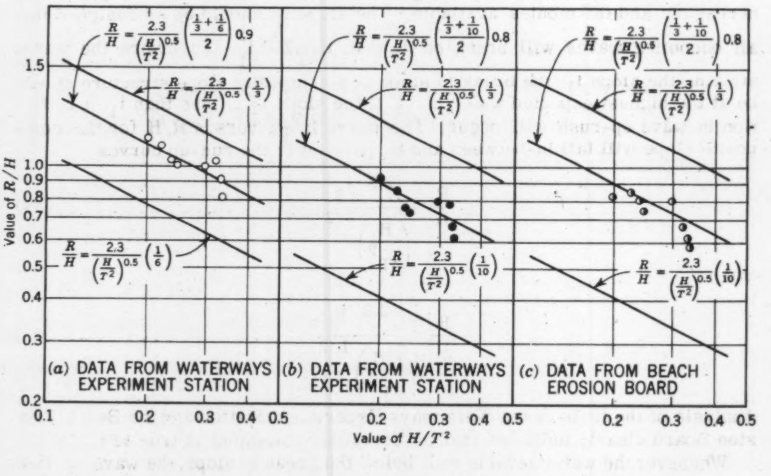


FIG. 10.—RATIO OF WAVE UP-RUSH TO WAVE HEIGHTS FOR COMPOSITE SLOPE STRUCTURES

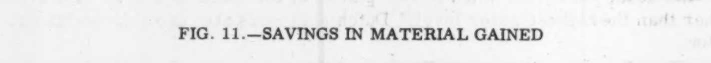


FIG. 11.—SAVINGS IN MATERIAL GAINED

The break in slope on a composite structure causes a further dissipation of wave energy over that expected with continuous slopes by creating additional turbulence. The amount of turbulence created depends on the angle between the slopes, that is, $\alpha_1 - \alpha_2$. The general formula for computing the up-rush on a composite slope when the water level is at the break in slope is

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} \left(\frac{\tan \alpha_1 + \tan \alpha_2}{2}\right) S \quad \ldots\ldots\ldots \quad (27)$$

in which S is a function of $(\alpha_1 - \alpha_2)$. In the special case in which α_1 equals α_2, Eq. 27 becomes identical with Eq. 21.

With reference to Fig. 10, curves were fitted by eye through the plotted points in accordance with Eq. 27. The obtained values of S were 0.9 for slopes 1:3 to 1:6 and 0:8 for slopes 1:3 to 1:10 and 1:10 to 1:3. It is interesting to note that the up-rush equation for the Waterways Experiment Station data on a composite structure of 1:3 to 1:10 was exactly the same as that for the Beach Erosion Board data on a composite structure of 1:10 to 1:3. This fact is extremely important, because, if the offshore conditions are such that the wave will definitely break on the lesser slope, a great savings in material can be effected. This is illustrated in Fig. 11, where the cross-hatched area represents the savings in material gained by using a 1:10 to 1:3 composite structure instead of a 1:3 to 1:10 structure.

Berms.—Whenever local conditions are such that it is impractical to extend the upper slope, i_2, of a composite structure so that all the oncoming wave energy is dissipated on this slope, it is necessary to resort to the use of a third slope, i_3, to retain the wave up-rush (Fig. 9). This triple-sloped structure is called a berm-type structure and the intermediary slope, i_2, is defined as the berm. The berm slope should be relatively flat, and, as previously studied, the break in slope between i_1 and i_2 should be at the highest expected storm water level. There has been much speculation in past years about what the berm width, B, should be and to what extent a berm reduces the wave up-rush.

The Hydraulics Laboratory at Delft suggests that $\frac{R}{H}$ is reduced by the factor $\left(1 - \frac{B}{L}\right)$ when a berm is used, in which $\frac{B}{L}$ is the berm width divided by the wave length. Analysis of tests which were conducted by the Waterways Experiment Station to determine the up-rush of berm-type structures revealed some interesting facts. The data contained in Fig. 12 are for structures in which i_1 is 1:3, i_2 is 1:20, and i_3 is 1:3, and the oncoming waves were such that they broke on slope i_1. The wave-heights and periods were kept constant; the only variable being the berm width, which had values of 30 ft, 50 ft, and 70 ft. The graphical analysis of Fig. 12 shows a reduction in wave up-rush with increasing values of $\frac{B}{L}$. The wave-lengths were not constant for each wave-type generated, but they did not vary appreciably. The average values of $\frac{B}{L}$ were .18, .31, and .43 for berm widths of 30 ft, 50 ft, and 70 ft.

The up-rush for the berm with the $\frac{B}{L}$ ratio of .43 was the same as for a composite slope of 1:3 to 1:20 computed by Eq. 27 with an S value of 0.8. The reduction in up-rush for the $\frac{B}{L}$ ratio of .18 was not appreciable enough to warrant the berm-type construction. Many more laboratory tests will have to be conducted before specific facts can be ascertained, but it may be said generally that the berm width should be such that $\frac{B}{L}$ is greater than 20%, that is

$$\frac{B}{L} \geq \frac{1}{5} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (28)$$

The effect of slope i_3 is one factor which needs clarification. The experiments of Bruun have shown that i_3 is relatively unimportant. In a series of tests performed at the Laboratorie Dauphinois in Grenoble on a model of the North Kent sea-wall in England, the optimum solution arrived at after performing what was probably the most exhaustive model study up to that time of the effects of berms, composite slopes, and the use of artificial roughness on wave up-rush, was to plan a vertical wall at the end of the berm. In the case of the North Kent sea-wall, there was a space limitation that would not allow the use of a wide berm. The use of a vertical wall at the termination of the berm is satisfactory in design if $\frac{B}{L} > \frac{1}{5}$, because the addition of a vertical wall or a steep slope (1:3 or 1:2) at the termination of the berm creates additional turbulence with the result that additional energy is dissipated. No satisfactory quantitative solutions can be given at this time as to the effect and design of the i_3 slope. Again, further laboratory investigations are necessary.

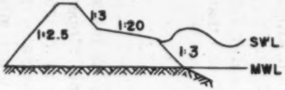
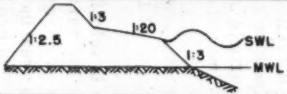
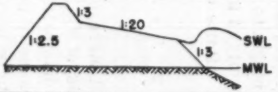
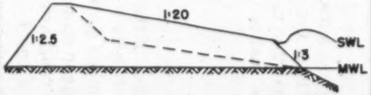
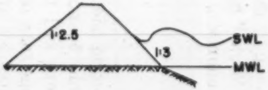
It is not difficult to visualize the great economy that may be obtained under certain conditions by the use of a berm. Berms are not cure-alls, but, when used judiciously, they can save a great deal of material and still ensure a safe structure.

To illustrate the choice of type of structure to use in the design of shore protective works, the following example is given. Utilizing the analyses of the results of the Waterways Experiment Station data shown in Fig. 12 and Eqs. 21 and 27, the wave up-rush has been computed for three berm-type structures, a composite slope structure, and a continuous slope structure. Each structure was designed to prevent overtopping by a 12-ft-high, 6-sec-period wave. The storm water level was assumed to be 5 ft above mean water level. The crown of the structure was designed 11 ft wide and the land slope was 1:2.5. The results of the analyses are shown in Table 4.

The composite slope has the lowest crest elevation but requires the most space. On the other hand, the continuous slope needs the minimum width but has the highest crest elevation. Surprisingly, the composite slope has either the largest or the least cross-sectional area, depending on whether the slopes were 1:3 to 1:20 or 1:20 to 1:3. The actual choice of design depends on the local problem. For instance, quite often local inhabitants do not want a structure that will cut off a pleasant view of the water, in which case the designer might have to resort to the structure with the lowest crest elevation. In this example, the 70-ft-wide berm and the composite slope structures had approximately the same wave up-rush and, therefore, the minimum crest elevation. In other words, a 4-ft-high vertical wall at the end of the 70-ft-wide berm had the same effect in reducing wave up-rush as 76 ft of additional 1:20 slope.

The steeper continuous slope compared favorably with the berm-type structures. However, up to this time, only smooth impermeable slopes have been

TABLE 4.—COMPARISON OF TYPES OF DESIGN STRUCTURES

Structure (1)	Sketch (2)	Wave data: H, in ft (3)	Wave data: T, in sec (4)	$\frac{H}{T^2}$ (5)	$\frac{R}{H}$ (6)	R, in ft above SWL (7)	Crest elevation (8)	Width (9)	Cross section area, in sq ft (10)
30 ft berm	 1:2.5 1:3 1:20 1:3 SWL MWL	12	6	.333	0.89	10.7	15.7	122.8	997
50 ft berm	 1:2.5 1:3 1:20 1:3 SWL MWL	12	6	.333	0.72	8.6	13.6	128.3	924
70 ft berm	 1:2.5 1:3 1:20 1:3 SWL MWL	12	6	.333	0.63	7.6	12.6	139.8	977
Composite	 1:2.5 1:20 1:3 SWL MWL	12	6	.333	0.61	7.3	12.3	202.8 (1:3 to 1:20) 163.7 (1:20 to 1:3)	1625 (1:3 to 1:20) 764 (1:20 to 1:3)
Continuous	 1:2.5 1:3 SWL MWL	12	6	.333	1.05	12.6	17.6	107.8	1045

presented. In the next two sections it will be shown that the slope of the structure is important when considering the dissipation of wave energy caused by roughness and permeability.

ROUGHNESS

It is known that the addition of roughness elements to a surface increases the turbulence and that this dissipates energy. Many structures have been built incorporating roughness in an attempt to reduce the wave up-rush; some have been successful and others have not. At the present time (1961), not enough data are available to make quantitative statements, but experience can be drawn on to give some qualitative hints.

The turbulence caused by the roughness elements transmits shear. The general equation for shear stress is of the type

$$\tau = \rho \; K \, V^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (29)$$

in which τ is the shear stress; ρ denotes the density of the liquid; K represents a factor depending on the roughness elements, their size and spacing; and V is the velocity near the bottom.

Much experimental work has been accomplished to determine the spacing of artificial roughness in the form of blocks which will give the maximum dissipation of energy. With reference to Fig. 13, J. W. Johnson has shown[20] that when $b/a = 4$, the maximum roughness coefficient is found when $c/a = 12$.

Little is known about the ratio of roughness height, a, to the wave height, H_o. Bruun has recommended[19] using roughness so that H_o/a is 3 to 4. It is of greatest importance that the roughness is of a size large enough to be "felt." In the experiments on the North Kent sea-wall, conducted by the Laboratorie Dauphinois, it was found that artificial roughness placed according to the results of Johnson decreased overtopping appreciably. It was proven in this case that small step-type roughness did not decrease the run-up to a great extent. A step-type roughness does not decrease the reflection, although it affects the run-up by the ratio of the step height to the wave height.

Miche[9] has analyzed the velocity of a particle of water along the bottom as an unbroken wave rushes up an infinite incline. He found that

$$V_{max} = \frac{H \; \pi}{\sin \alpha} \sqrt{\frac{g}{L}} \quad \ldots\ldots\ldots\ldots \quad (30)$$

To give one an idea of the great increase in velocity as the slope diminishes, Table 5 shows[9] relative values of V_{max} for various slopes for a given γ_o. There will be a much greater dissipation of energy by roughness on a gentle slope than on a steep slope because of the great increase in the water particle velocities and, if roughness is to be used, it is much more effective on a gentle slope. The use of roughness on a berm of sufficient width would undoubtedly be of great aid in reducing up-rush. But, because of the complex nature of the problem, it is highly recommended that any design incorporating artificial

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[20] "Rectangular Artificial Roughness in Open Channels," by J. W. Johnson, Transactions, Amer. Geophysical Union, May, 1945, pp. 906-913.

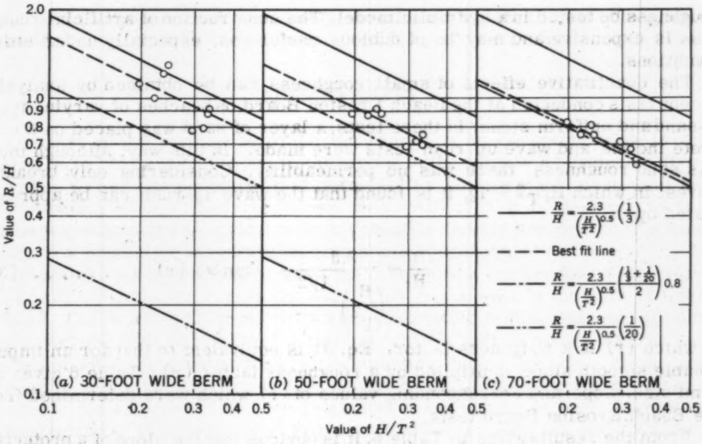

FIG. 12.—RATIO OF WAVE UP-RUSH TO WAVE HEIGHT FOR BERM TYPE STRUCTURES OF VARYING BERM WIDTHS

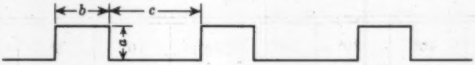

FIG. 13.—RECTANGULAR-TYPE ARTIFICIAL ROUGHNESS

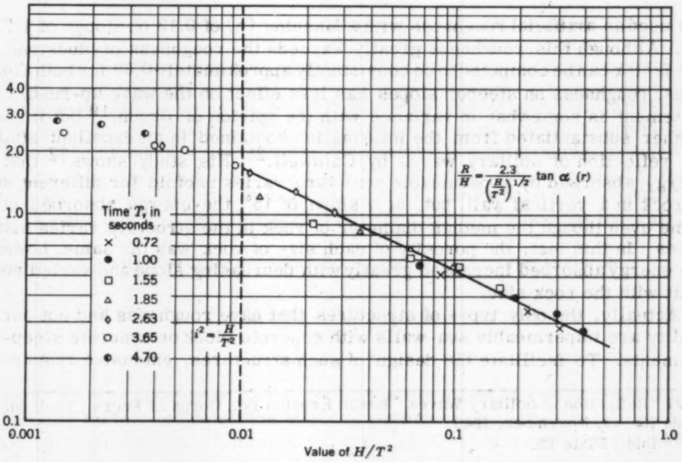

FIG. 14.—RATIO OF WAVE UP-RUSH TO WAVE HEIGHTS FOR A CONTINUOUS ROUGH PERMEABLE SLOPE OF 1:10

roughness be tested in a hydraulic model. The construction of artificial roughness is expensive and may be of dubious usefulness, especially under storm conditions.

The quantitative effects of small roughness can be obtained by analyzing recent tests conducted at the Beach Erosion Board on beaches of varying types of sand and uniform stone. In these tests, a layer of sand was placed on a concrete incline and wave up-rush tests were made. In this way, although there was sand roughness, there was no permeability. Considering only breaking waves, in which $H/_{T}2 > i_2$, it is found that the wave up-rush can be approximated by

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} (\tan \alpha)\ (r) \quad \ldots\ldots\ldots \quad (31)$$

in which (r) is a roughness factor. Eq. 31 is equivalent to that for an impermeable smooth slope multiplied by a roughness factor (r). Table 6 gives the sand size, slope, and corresponding values of (r) which were determined from the Beach Erosion Board tests.

From the results given in Table 6, it is obvious that the slope of a protective structure or a beach plays an important part in the effect of roughness. Sibul[18] has used an artificial roughness with a Manning (n) of 0.13 on slopes of 1:2 and 1:3. Although this roughness greatly exceeds the roughness of bluestone, the (r) factor can be computed to be consistently approximately 0.80 for both slopes. Thus, roughness on steeper slopes has less effect on the wave up-rush. This statement is somewhat in variance with the opinion of Bruun,[19] but it can be further substantiated from the information contained in an excellent study of the reflection of solitary waves by Caldwell.[21] His study shows[22] that the energy absorbed by a permeable structure varies twofold for different sizes of rock in a vertical wall, but, at a slope of 15°, the energy absorbed is the same even though the median diameter of rock in the structure varies sixteen times. In this test, the porosity of each size of rock was the same, however, the energy absorbed increased greatly with decreasing slope and varied somewhat with the rock size.

TABLE 5.—COMPUTED VALUES OF V_{max} FOR GIVEN SLOPES, FROM MICHE[8]

α	90°	45°	30°	18°	15°	10°	5°	2°	1°
V_{max}	1	2	3.46	7.25	9.45	17.2	48.5	192	542

Actually, the only types of structures that have roughness and not permeability are impermeable sea-walls with concrete block or concrete steep-type elements. To facilitate the design of such structures, extensive research is necessary to compute the roughness factors for the various types of roughness elements.

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[21] "Reflection of Solitary Waves," Beach Erosion Bd., Corps of Engrs., Tech. Memoranda No. 11, November, 1949.

[22] Ibid., Plate 13.

PERMEABILITY

The last major method by which the energy of an oncoming wave is dissipated is by heat generated due to the mixing in the voids of a permeable structure. The writer has concluded that permeability plays a much more vital role in dissipating energy than roughness does. Although it is possible to have roughness without permeability, it is not possible to have any degree of permeability in a protective structure without some roughness.

Laboratory investigations of wave up-rush show a tremendous reduction in run-up when porous structures are used. This can be seen in the work of Grantham.[13] The work of Caldwell[21] is also interesting in this respect, although it is concerned with reflection of a solitary wave. Caldwell shows that the percentage of wave energy absorbed by the structure varies linearly with the porosity up to values of porosity of 50%, at which time, 90% of the wave energy is absorbed. He also shows that the thickness of the structure is of importance in the absorption of wave energy. For a vertical permeable sea-wall, Caldwell finds that there is no further absorption of incident wave energy for values of $W/D > 2$, in which W/D is the wall thickness divided by the water depth.

TABLE 6.—ROUGHNESS FACTORS FOR VARYING SAND SIZES AND BEACH SLOPES

Sand size (1)	(r) slope 1:10 (2)	(r) slope 1:30 (3)
Smooth	1.00	1.00
0.2 mm	0.96	0.89
1.0 mm	0.85	0.78
2.0 mm	0.82	0.71
3.4 mm	0.76	0.64
Blue Stone	0.70	0.49

The reduction in wave up-rush attributed to roughness and permeability can also be determined from the tests of the Beach Erosion Board. As previously explained, the effect of roughness was obtained by placing granular beach material in a thin layer on a concrete slope. The effect of roughness and permeability together was obtained by eliminating the concrete base and constructing the whole slope of the granular material. The writer found that the ratio of wave up-rush to wave-height for waves breaking on a rough and porous continuous slope could again be determined by the general-type formula

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} \tan \alpha \; (r) \; (p) \; \ldots\ldots \quad (32)$$

in which (r), the roughness factor, indicates the effect of roughness and (p), the porosity factor, indicates the effect of permeability.

The fact that in Eqs. 31 and 32 the ratio of wave up-rush to wave-height varies directly with the roughness factor (r) or the roughness factor and the porosity factor (r) (p) appears to over-simplify a complex phenomenon. However, actual results indicate that this is the case (Fig. 14), where the experimental data closely follow the curve

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} \left(\frac{1}{10}\right) \quad (.74) \ldots\ldots\ldots (33)$$

The reduction of wave up-rush attributed to roughness and porosity has been computed from the Beach Erosion Board data and the values of the factors (r) and (p) are listed in Table 7.

Note the great effect of permeability when bluestone is used. It is unfortunate that the actual porosities of the materials used, are not available but it was visually apparent that the bluestone had a much greater porosity than the sand. The run-up was reduced 54% because of permeability alone.

TABLE 7.—POROSITY FACTORS FOR VARYING SAND SIZES AND BEACH SLOPES

Sand size (1)	Non-porous (r) (2)	Porous (r)(p) (3)	(p) (4)
(a) Slope 1:10			
Smooth	1.00	1.00	1.00
0.2 mm	0.96	0.91	0.95
1.0 mm	0.85	0.82	0.96
2.0 mm	0.82	0.74	0.90
3.4 mm	0.76	0.65	0.85
Blue Stone	0.70	0.38	0.54
(b) Slope 1:30			
0.2 mm	0.89	0.77	0.86

From the one value available on the 1:30 slope, it appears that the reduction of wave up-rush due to permeability is greater on lesser slopes. This is to be expected.

There is no doubt that a porous structure greatly reduces wave up-rush. However, extreme caution should be exercised when considering the use of porous material, because such material is subject to wave attack and the uplift pressures caused by receding waves. In Holland, the writer has seen huge rocks which had been lifted out of place by uplift pressures. Again, in England, the writer has seen the force of a wave striking a rubble sea-wall tear it apart. The stability of a rubblemound breakwater or porous sea-wall should be tested in the laboratory.

It may be many years before systematic laboratory investigations will give conclusive quantitative results concerning the actual effects of various types of construction material on wave up-rush. As previously stated, it is unusual, except in concrete block or step-type artificial roughness, to find a roughened structure without porosity. Therefore, the roughness factor (r) and the porosity factor (p) can be combined into a single reduction factor (ϕ) in which

$$\phi = (r)\ (p) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (34)$$

The ratio R/H theoretically should be a function of $\tan \alpha \left(\frac{H}{T^2}\right)^{-1/2}$. The general formula for computing the wave up-rush on a porous, rough continuous slope is therefore

$$\frac{R}{H} = 2.3 \; \frac{\tan \alpha}{\left(\frac{H}{T^2}\right)^{1/2}} \; (\phi) \quad \ldots\ldots\ldots \quad (35)$$

Thus, whenever rough, porous material is used in the construction of shore protective works, the reduction factor ϕ will measure the effectiveness of the material in dissipating the oncoming wave energy. The most effective material is that which has the least reduction factor. Table 7 shows that the reduction factor for bluestone on a 1:10 slope is 0:38. This demonstrates that bluestone on the 1:10 slope reduced the wave up-rush 62% more than what it would have been on a smooth impermeable slope. Consequently it was highly effective. The results of laboratory tests indicate that the up-rush of surging waves is reduced by roughness and permeability in the same ratio as the up-rush of breaking waves, that is, by the reduction factor, ϕ.

In the past few years, the tetrapod design of the Laboratorie Dauphinois has become popular in Europe for harbor breakwaters. The tetrapods present a rough surface and have a porosity, according to Waterways Experiment Station, of approximately 50%. They can be constructed in any size, and, therefore, in any weight, and because of their interlocking qualities are very stable.

An example was given in Table 4 illustrating the types of structures which can be used in the design of shore protective works. A 1:13 continuous slope structure compared favorably with a 70-ft-wide berm-type structure when both structures had impermeable slopes. From the analyses of the effects of roughness and permeability on the dissipation of energy, it should be realized that not only will an appreciable reduction in up-rush occur with the use of rough and permeable structures, thus conserving space and material, but that the effects of roughness and permeability are most pronounced in reducing wave up-rush on gentle sloping structures. Therefore, it is doubtful if the 1:3 continuous slope structure would have compared favorably with a berm-type structure that judiciously used roughness and porosity.

CONCLUSIONS

The conclusions expressed were obtained by analyses of hydraulic laboratory model tests. There was always the possibility of scale effect when utilizing model results. Furthermore, the waves utilized were mechanically generated, as opposed to wind-generated waves which one encounters in nature. Consequently, there was no variation in wave attack such as was found in nature due to the diversity of waves that make up a wave train. For design purposes, the wave characteristics utilized may be considered as those of the significant wave, or the average height and period of the one-third highest waves of a given wave train.

Research on the subject matter is continuing; much of it is being conducted with wind-generated waves and large-scale wave tanks. It is the opinion of the writer that many of the concepts presented herein are fundamental and will not vary, although derived constants may change in large-scale model studies or with wind-generated waves.

The following salient features of the design of coastal protective structures were studied:

1. If at all possible, wave reflection should be kept to a minimum.
2. To minimize wave reflection, the slope of a protective structure should be such that $i^2 < H/_T 2$.
3. The wave up-rush on a continuous sloped impermeable structure can be computed by Eq. 21 for $i^2 < H/_T 2$ and $H \approx H_o$.
4. For surging storm waves, the wave up-rush to be used for design purposes can be approximated by Eq. 22 for $i^2 > H/_T 2$ and $H \approx H_o$.
5. The wave up-rush on composite slopes where the storm water level is at the break in slope can be computed by Eq. 27 for $i_1^2 < H/_T 2$; $H \approx H_o$; and $S \approx .8$ to $.9$.
6. Berms will reduce wave up-rush. The berm width should be such that $B/L \geq 1/5$. The use of a vertical wall at the termination of the berm has proven satisfactory in dissipating energy.
7. Artificial roughness is much more effective on gentle slopes than on steep slopes. The roughness reduces the wave up-rush on a continuous sloping structure according to Eq. 31 for $i^2 < H/_T 2$ and $H \approx H_o$.
8. Eq. 35 is the general equation for computing the wave up-rush on a porous, rough, continuous slope. The reduction factor ϕ, is a measure of the effectiveness of the material in dissipating the oncoming wave energy.
9. Tetrapods, or a similar-type material, are ideal construction material because they incorporate both roughness and porosity, have interlocking qualities, and can be constructed in any size.

APPENDIX.—APPLICATION OF RESULTS TO CENTRAL AND SOUTHERN FLORIDA PROJECT

The design of the levees at Lake Okeechobee is currently of great interest to the Office of the Chief of Engineers. The writer understands that the redesign of the levees has considered the use of an impervious continuous slope of 1:3 or less or a composite impervious slope of approximately 1:10 to 1:3. The wave steepnesses of storm waves in shallow water are generally greater than one twenty-fifth, $\gamma_o > .04$. The critical slope is defined as

$$i = \sqrt{5.12\ \gamma_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (36)$$

so that for the design slopes contemplated and the expected wave steepnesses, the waves will break on the levees. Therefore, the wave up-rush can be computed by either Eq. 21 or Eq. 27.

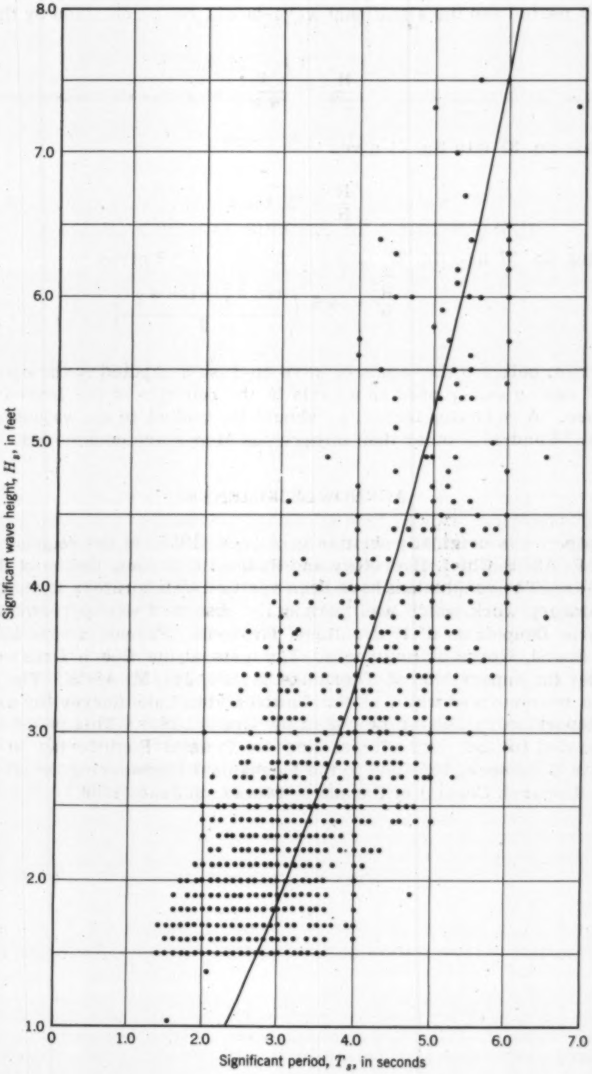

FIG. 15.—RELATIONSHIP BETWEEN WAVE HEIGHT AND WAVE PERIOD, LAKE OKEECHOBEE

Fig. 15 shows[4] that a good approximation of the relationship between the significant period and the significant wave-height can be obtained by the parabola

$$\frac{H}{T^2} = \frac{1}{4.8} \qquad (37)$$

Substituting Eq. 37 into Eq. 21 gives

$$\frac{R}{H} = 5 \tan \alpha \qquad (38)$$

Substituting Eq. 37 into Eq. 27, using an S value of 0.9 gives

$$\frac{R}{H} = 4.5 \left(\frac{\tan \alpha_1 + \tan \alpha_2}{2} \right) \qquad (39)$$

The writer believes the values of wave up-rush computed by the use of Eqs. 38 and 39 can be safely used as a basis in the redesign of the levees at Lake Okeechobee. A reduction factor, ϕ, should be applied to the values obtained from Eqs. 38 and 39 if other than impervious slopes are considered.

ACKNOWLEDGMENTS

This paper was originally written in August, 1955, at the request of A. L. Cochran, F. ASCE, Chief, Hydrology and Hydraulic Branch, Office of the Chief of Engineers. The conclusions have been verified by laboratory investigations. The laboratory work which was personally observed was performed in the Laboratorie Dauphinois d' Hydraulique, Grenoble, France, and in the Beach Erosion Board, Corps of Engineers. The tests at the Beach Erosion Board were under the supervision of Thorndike Saville, Jr., M. ASCE. The original paper has been revised while at the United States Lake Survey for use in the Survey Report on the Water Levels of the Great Lakes. This paper has been communicated to the Sixth Conference on Coastal Engineering at Miami, Florida, in December, 1957 and to the Mechanical Engineering Division of the National Research Council of Canada at Ottawa, in June, 1958.

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CASE OF CRITICAL SURGING OF A MOORED SHIP

By Basil W. Wilson,[1] F. ASCE

With Discussion By W. C. Q. Joosting; and Basil W. Wilson

SYNOPSIS

An analysis is made of the circumstances in which an oil tanker, berthed at a solid quay wall in a rectangular basin of a port subject to influences of long period waves, developed critical surging motions and broke adrift after rupture of its mooring lines. The response characteristics of the ship are examined in the light of the non-linear equation of motion for longitudinal surging and the best available information on the conditions of mooring, the state of the sea and the time of the tide. From the location of the ship in the dock it is inferred that the possible modes of oscillation of the water body in the harbor basin that could have influenced the ship would have been the uninodal, binodal, quinquinodal (and possibly sextinodal) seiches. It is then shown that with the rise of tide in the dock and the accompanying known and probable amplitude increases of the various modes of seiches, only the quinquinodal seiche with a period of 22 sec could have been responsible for the development of rope tensions sufficient to fracture the mooring lines.

INTRODUCTION

In harbors that are exposed to large tracts of ocean along lines of approach of cyclonic and frontal storms, ingress of long period waves, originating with

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such storms, sometimes excites oscillations of the water masses (seiches) in the basins and gives rise to troublesome motions of moored ships.[2]

The behavior of a ship at a berth is often closely linked with its location in the dock with respect to the nodes of the water oscillations, with the state of the tide and the manner of the ship's mooring (tightness or slackness, number and elasticity of its mooring lines), with its size and draft (light or loaded), and of course, with the magnitude of whatever seiches may be prevalent during occupancy of the berth. The combination of these factors sometimes produces unexpected results in that a ship in an unfavorable position may exhibit critical surging, sway or yawing motions in relatively mild sea conditions (to which neighboring ships might be almost immune), whereas another ship in the same berth on a different occasion may lie relatively quiet and unaffected in considerably greater sea disturbances. The peculiarities of such occurrences led the writer[3] to attempt an analysis of the idealized motions of a moored ship located at the node of a seiche. The emphasis was on longitudinal surging that has since been further considered theoretically by Abramson and the writer[4] and successively by Russell,[5] Joosting,[6] O'Brien and Kuchenreuther[7,8] and Wilson[9,10] again. In the course of these presentations several errors in the original formulation of the problem have been corrected and the theory now seems capable of explaining the apparently enigmatic behavior-patterns of ships previously referred to. It is the purpose of the present paper to use this theory in explanation of the circumstances in which a tanker of 18,000 dead-weight tons broke adrift from its moorings in a harbor basin.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically for convenience of reference in the Appendix.

CASE HISTORY

The tanker, of overall length 480 ft, beam $a = 66.1$ ft and mean loaded draft $D = 27.87$ ft, entered port and docked at low tide in the early hours of May 5, 1952, along the solid vertical quay wall at one end of a long rectangular basin.

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[2] "Origin and Effects of Long Period Waves in Ports," by B. W. Wilson, Sect. II, Communication 1, XIXth Internatl. Navigation Congress, London, July, 1957, p. 13.

[3] "Ship Response to Range Action in Harbor Basins," by B. W. Wilson, Transactions, ASCE, Vol. 116, 1951, p. 1129.

[4] "A Further Analysis of the Longitudinal Response of Moored Vessels to Sea Oscillations," by H. N. Abramson and B. W. Wilson, Proceedings, Mid Western Conf. on Fluid and Solid Mechanics, Purdue, Ind., September, 1955.

[5] "Origin and Effects of Long Period Waves in Ports," by G. E. R. Deacon, R. C. H. Russell, and J. E. G. Palmer, Sect. II, Communication 1, XIXth Internatl. Navigation Congress, London, July, 1957, p. 75.

[6] "Investigation into Long-Period Waves in Ports," by W. C. Q. Joosting, Sect. II, Communication 1, XIXth Internatl. Navigation Congress, London, July, 1957, p. 205.

[7] "Forces Induced on a Large Vessel by Surge," by J. T. O'Brien and D. I. Kuchenreuther, Proceedings, ASCE, Vol. 84, No. WW2, March, 1957.

[8] "Forces Induced by Waves on the Moored USS Norton Sound (AVM-1)," by J. T. O'Brien and D. I. Keutchenreuther, Tech. Memorandum M-129, U. S. Navy Civ. Engrg. Lab., Port Hueneme, Calif., April, 1952.

[9] Discussion by B. W. Wilson of "Forces Induced on a Large Vessel by Surge," by J. T. O'Brien and D. I. Kuchenreuther, Proceedings, ASCE, Vol. 84, No. WW5, December, 1958, p. 1884.

[10] "The Energy Problem in the Mooring of Ships Exposed to Waves," by B. W. Wilson, Proceedings, Princeton Conf. on Berthing and Cargo Handling in Exposed Locations, October, 1958, p. 1; Bulletin Permanent International Assoc. of Navigation Congresses, No. 50, 1959.

The ship was breasted off from the fixed timber waling on the quay by conventional timber floating fenders and tied by the system of mooring lines shown in Fig. 1. The location of the ship in the 2100 ft width of dock (Fig. 2) was such that its mid-ship distance b from the long side of the dock, nearest its bow, was 590 ft.

The rectangular basin was dimensionally susceptible to several possible modes of transverse oscillation, shown in Fig. 2. The existence of such seiches, on occasion, in the port concerned has been established and their natural periods τ can be computed with a fair degree of accuracy from the equation[11]

$$\tau = \frac{2\,B}{m\,\sqrt{g\,d}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which B is the width of the basin, m an integer (1, 2, 3 . . .) defining the harmonic mode, g the acceleration due to gravity and d the uniform water depth. For the water depths prevalent at the time of docking of the ship and 6 hr later at high tide, the values of τ are found to be as given in Table 1. Period differences at the two states of tide are minor so that it is possible to represent the nodal configurations of the individual modes of water oscillation by the series of end views shown in Figs. 2(a) to 2(g).

TABLE 1.—PERIODS OF TRANSVERSE SEICHES

Harmonic Mode m (1)	On Docking (Low Tide)		High Tide	
	Water depth, d, in feet (2)	Seiche Period, τ, in seconds (3)	Water depth, d, in feet (4)	Seiche Period, τ, in seconds (5)
1	44	111.5	46	109.1
2	44	55.7	46	54.5
3	44	37.2	46	36.4
4	44	27.9	46	27.3
5	44	22.3	46	21.8
6	44	18.6	46	18.2
7	44	15.9	46	15.6

From available tide records only the amplitude of the fundamental uninodal seiche was known with reasonable accuracy. This is shown in Fig. 7(b) and forms the basis for the conjectural magnitudes of the binodal and quinquinodal seiches, also shown. The corresponding state of the tide is reflected in Fig. 7(a). It will be seen from Fig. 7(b) that seiche amplitudes at the time that the tanker docked were quite small but that they rapidly increased on the rising tide. In the interval shown shaded in Figs. 7(a) and 7(b) the surging of the ship resulted in her breaking adrift. Attempts to remoor her failed even with the assistance of two powerful tugs brought in between 1100 and 1130 to force the ship abreast of the quay. Finally the tanker was pulled clear and removed

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[11] Hydrodynamics, by H. Lamb, Cambridge Univ. Press, England, 1932 edition.

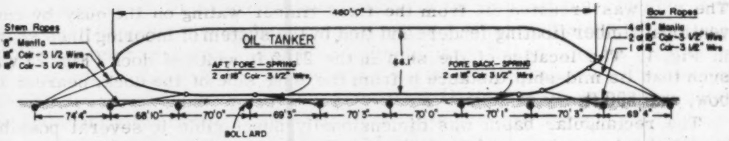

480'-0"
Stem Ropes
4 of 8" Manila
2 of 18" Coir - 3 1/2 Wire
1 of 18" Coir - 3 1/2 Wire
OIL TANKER
AFT. FORE - SPRING
2 of 18" Coir - 3 1/2" Wire
66.1'
FORE BACK - SPRING
2 of 18" Coir 3 1/2" Wire
Bow Ropes
4 of 8" Manila
2 of 18" Coir-3 1/2" Wire
1 of 18" Coir-3 1/2" Wire
74'4"
68'10"
70'0"
69'3"
70'3"
70'0"
70'1"
70'3"
69'4"
BOLLARD

FIG. 1.—SYSTEM OF MOORING-LINES USED FOR SECURING OIL-TANKER AT BERTH

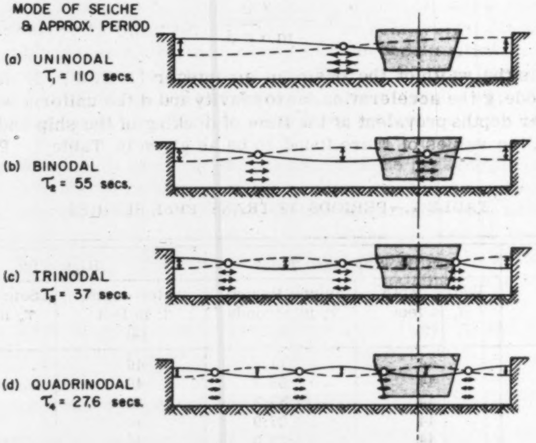

MODE OF SEICHE
& APPROX. PERIOD
(a) UNINODAL
T_1 = 110 secs.
(b) BINODAL
T_2 = 55 secs.
(c) TRINODAL
T_3 = 37 secs.
(d) QUADRINODAL
T_4 = 27.6 secs.
(e) QUINQUINODAL
T_5 = 22 secs.
(f) SEXTINODAL
T_6 = 18.4 secs.
(g) SEPTUANODAL
T_7 = 15.7 secs.
b
B
PLAN VIEW

FIG. 2.—LOCATION OF OIL-TANKER IN DOCK IN RELATION TO SEVEN HARMONIC MODES OF TRANSVERSE OSCILLATION OF THE WATER BODY

to the open roadstead for safety. It was not until almost 18 hr later [Fig. 7(b)] that the disturbances had abated sufficiently for the ship to redock.

DIMENSIONS OF THE SHIP MOORING LINES

In order to estimate the behavior of the ship in surging it is necessary to make some initial assumption in regard to the lengths S_0, Fig. 3(b), that the individual mooring lines would have had at the transient moment of the ship's passing through the equilibrium position it would occupy in the absence of any disturbance. This entails some consideration of mooring line geometry. In the equilibrium position, A, Figs. 3(a) and 3(b), the horizontal projection of a typical bow-rope, AB, of length S_0, is L_0, its horizontal angle of inclination to the dock front, θ, and its vertical projection above the quay level, H. In displacing the longitudinal distance X along the quay from A to A' [Fig. 3(a)], the horizontal projection of the rope is increased by dL to L and the corresponding length of the rope, allowing for extension under stress, from S_0 to S. It is convenient, tentatively, to regard the rope situations AB and A'B as disposed in the same vertical plane, as shown in Fig. 3(b).

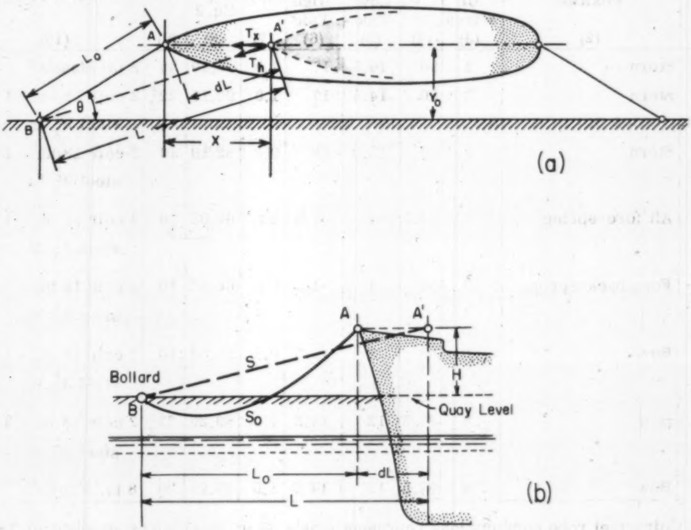


FIG. 3.—MOORING LINE GEOMETRY: (A) PLAN VIEW OF SHIP SHOWING STERNWARD SURGE TRANSLATION X; (B) SIDE ELEVATION VIEW OF SHIP PROJECTED IN THE VERTICAL PLANE OF THE ROPES

The dimension S_0 will obviously be greater than the direct length A'B $\left(= \sqrt{L_0^2 + H^2}\right)$. Because the tanker was moored during a period of slight surging at low tide it is almost certain that the berthing crew would have attempted to make the ropes as tight as possible by taking advantage of each lunge of the ship forward or aft. Hence, the probability is that the dimension S_o can be taken to exceed $\sqrt{L_o^2 + H^2}$ by only a small amount, ΔS_o, estimated at 1 ft for locations 1, 2, 7 and 8, 0.8 ft for locations 3 and 6, and 0.6 ft for locations 4 and 5 (Table 2). For all the ropes shown in Fig. 1, therefore, S_o has been determined from

TABLE 2.—MOORING ROPE DIMENSIONS

Item No. (1)	Mooring Line Position (2)	No. of Ropes or Wires (3)	Dimensions, in feet							Rope Type[a] (10)
			L_o (4)	H		ΔS_o (7)	S_o Eq. 2 (8)	θ° (9)		
				Low Tide (5)	High Tide (6)					
1	Stern	4	90	14.5	17	1.0	92.10	30		8 in. Manila[b]
2	Stern	2	90	14.5	17	1.0	92.10	$22\frac{1}{2}$		2 coir 18 in. - 1 steel $3\frac{1}{2}$ in.
3	Stern	1	80	13.5	16	0.8	82.10	10		2 coir 18 in. - 1 steel $3\frac{1}{2}$ in.
4	Aft fore-spring	2	67.5	-4	-1.5	0.6	68.02	10		2 coir 18 in. - 1 steel $3\frac{1}{2}$ in.
5	Fore back-spring	2	60	4	-1.5	0.6	60.57	10		2 coir 18 in. - 1 steel $3\frac{1}{2}$ in.
6	Bow	1	65	8	10.5	0.8	66.34	10		2 coir 18 in. - 1 steel $3\frac{1}{2}$ in.
7	Bow	2	87.5	12	14.5	1.0	89.20	$22\frac{1}{2}$		2 coir 18 in. - 1 steel $3\frac{1}{2}$ in.
8	Bow	4	87.5	12	14.5	1.0	89.20	30		8 in. Manila[b]

[a] Coir-steel rope combinations represent single $3\frac{1}{2}$ in. steel wires attached to 18 ft length loops of 18 in. coir rope

[b] The four 8 in. manila ropes are assumed elastically equivalent to item 3 (or 6)

$$S_o = \Delta S_o + \sqrt{L_o^2 + H^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

for the values of ΔS_o, L_o and H, estimated from available information, along with values of θ, in Table 2.

ELASTIC PROPERTIES OF THE MOORING ROPES

The length S assumed by a rope under the tension necessary to restrain the longitudinal movement of the ship to an amount X [Figs. 3(a) and 3(b)] will exceed S_o by the amount of stretch dS. Because a rope under strong tension will be pulled effectively into straight line configuration,[3] it follows readily that the increase in horizontal projection, d L, is determinable from

$$dL = \sqrt{S^2 - H^2} - L_o \quad \dots\dots\dots\dots\dots\dots \quad (3)$$

In terms of dS, then

$$dL = \sqrt{(S_o + dS)^2 - H^2} - L_o \quad \dots\dots\dots\dots\dots \quad (4)$$

From Fig. 3(a) it will be seen that the longitudinal component, T_x of the horizontal component T_h of the tension in the rope at the ship is given by

$$T_x = T_h \left[1 - \left(\frac{Y_o}{L}\right)^2\right]^{\frac{1}{2}} \quad \dots\dots\dots\dots\dots \quad (5)$$

in which Y_o is the distance of the center line of the ship from the quay. Further, because the longitudinal ship movement X is

$$X = L\left[1 - \left(\frac{Y_o}{L}\right)^2\right]^{\frac{1}{2}} - L_o\left[1 - \left(\frac{Y_o}{L_o}\right)^2\right]^{\frac{1}{2}} \quad \dots\dots\dots \quad (6)$$

we may perform the binomial expansions of the roots to two terms with sufficient accuracy, having regard to the values of the fractions Y_o/L and Y_o/L_o $(< 2/3)$ and thus obtain

$$X = (L - L_o)\left[1 + \frac{1}{2}\,\frac{Y_o^2}{LL_o}\right] \quad \dots\dots\dots\dots\dots \quad (7)$$

Dividing Eq. 7 by Eq. 5 yields

$$\frac{X}{T_x} = \frac{dL}{T_h}\left[\frac{1 + \frac{1}{2}\left(\frac{Y_o^2}{LL_o}\right)}{1 - \frac{1}{2}\left(\frac{Y_o}{L}\right)^2}\right] \quad \dots\dots\dots\dots\dots \quad (8)$$

This result can be further simplified, without serious error, to

$$\frac{X}{T_x} = \frac{dL}{T_h}\left[1 + \left(\frac{Y_o}{L_o}\right)^2\right] \quad \dots\dots\dots\dots\dots \quad (9)$$

that may then be written

$$\frac{X}{T_x} = \frac{dL\left(1 + \sin^2\theta\right)}{T_h} \quad \dots\dots\dots\dots\dots\dots \quad (10)$$

It can be shown that the difference between T_h and T, the full rope tension, is negligibly small even at comparatively low tensions.[3] Eq. 10, therefore, shows that the desired functional relationship between rope restraint and ship movement in the logitudinal direction can be obtained directly from the relationship between $dL\left(1 + \sin^2 \theta\right)$ and T.

It will be seen from Eq. 4 that this relationship really resolves itself into knowing the dependence of rope extension dS on tension T, that must be found experimentally under conditions of repeated loading. Typical test results for 3 1/2-in. steel wire and 18-in. coir rope, obtained by plotting the center-lines of the repeated-loading hysteresis-loops from load-extension diagrams, are shown in Fig. 4(a).

By suitably choosing values of T in increasing increments and allowing for the composite lengths of coir and wire rope in the coir-steel combinations (see footnote, Table 2), the corresponding values of $dL\left(1 + \sin^2 \theta\right)$ may be computed for each mooring line from Eq. 4, Table 2 and Fig. 3(a), with results shown in Fig. 4(b). These apply strictly to the several ropes leading aft; details for the forward leading ropes have not been evaluated as computational probes suggested that no important differences would be found.

From Fig. 4(b) the cumulative sum of the tensions in all the ropes, ΣT_x, for any given value of X, can readily be obtained. Plotted against X in Fig. 5 the results yield the desired relationships between total restoring force R_x and ship movement X in the longitudinal direction. Because test results suggest that worn 18 in. coir rope is liable to failure at or near 15 tons, the capacity of the coir loops or 'springs' is not much above 30 tons. Accordingly Fig. 4(b) shows that the backsprings, item 4 of Table 2, are liable to failure when the amplitude of ship movement reaches about 2 ft. The limits of failure are indicated in Fig. 5.

TABLE 3.—SPRING CHARACTERISTICS OF MOORING LINES

Time on May 5, 1952 (1)	Tidal Condition (2)	Water Depth d, in feet (3)	Spring Characteristics C, (Tons - ft.$^{-n}$) (4)	Spring Characteristics n (5)
0700	Low	44	10.0	3.3
1200	High	46	26.1	2.4

For convenience of formulation of the equation of surge motion of the ship the curves of Fig. 5 have been fitted by an equation of the type

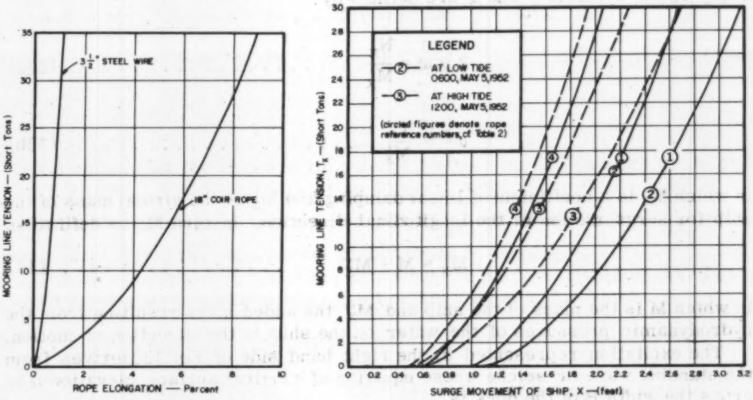
$$R_x = C X^n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

in which C is a constant and n a numerical exponent. The appropriate values of C and n found to give the best fit of Eq. 11 to the curves, for relatively large values of R_x, are given in Table 3 (see Fig. 5).

SURGE MOTION OF THE SHIP UNDER MOORING LINE RESTRAINTS

The equation of motion of the ship in the transverse standing wave system indicated in Fig. 2 has been developed elsewhere[10] and will merely be stated here in the form

$$\ddot{X} + 2\beta\omega\dot{X} + \omega^2 X^n = \xi\left(\sigma \sin\phi - 2\beta\omega\cos\phi\right) \quad . . \quad (12)$$



(a) MEAN TENSION-ELONGATION RELATIONSHIPS UNDER REPEATED LOADING

(b) TENSION-SURGE MOVEMENT RELATIONSHIPS FOR INDIVIDUAL MOORING LINES

FIG. 4

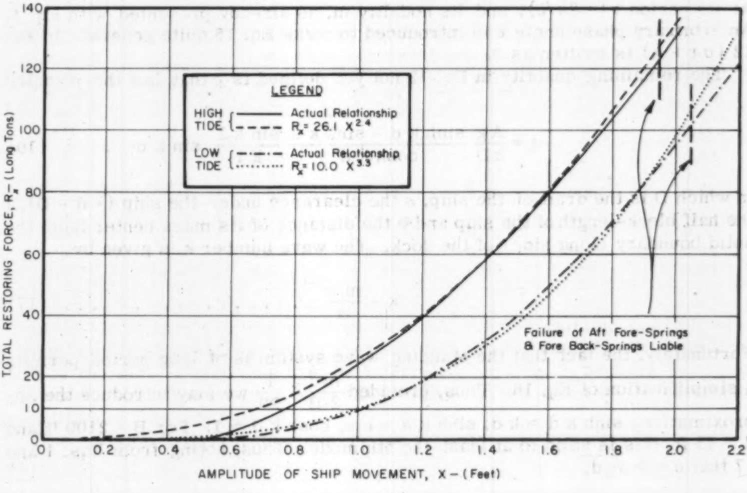


FIG. 5.—TOTAL RESTORING FORCE RELATIONSHIP TO SURGE-TRANSLATION OF THE SHIP

In Eq. 12 the symbols β and ω are defined by

$$2\,\beta\,\omega = \frac{N_X}{M_X'} \qquad \text{(13a)}$$

$$\omega^2 = \frac{C}{M_X'} \qquad \text{(13b)}$$

in which N_X is a coefficient of linear damping and M_X' is the virtual mass of the ship for surge motion in the longitudinal direction. In turn M_X' is defined as

$$M_X' = M + M_X'' \qquad \text{(14)}$$

in which M is the mass of the ship and M_X'' the added mass resulting from the hydrodynamic pressures of the water on the ship in the direction of motion.

The excitation represented by the right hand side of Eq. 12 derives from the standing wave or seiche whose equation of vertical surface elevation η across the width B of the dock is

$$\eta = A \cos \frac{m\,\pi\,x}{B} \cos(\sigma t + \epsilon) \qquad \text{(15)}$$

referred to an origin in the still water surface at one long side of the dock and an x-axis in the water plane traverse thereto. The amplitude of the seiche is A; its period $\tau\,(= 2\pi/\sigma)$; and its nodality m, as already presented with Eq. 1. An arbitrary phase angle ϵ is introduced to make Eq. 15 quite general. In Eq. 12 $(\sigma t + \epsilon)$ is written as ϕ.

The remaining quantity in Eq. 12 not yet defined is ζ that has the meaning

$$\zeta = \frac{A g}{\sigma D} \; \frac{\sinh k d - \sinh k s}{\cosh k d} \; \frac{\sin k l}{k l} \; \sin k b \qquad \text{(16)}$$

in which D is the draft of the ship, s the clearance under the ship (= d - D), l the half block-length of the ship and b the distance of its mass center from the solid boundary (long side) of the dock. The wave number k is given by

$$k = \frac{m\,\pi}{B} \qquad \text{(17)}$$

Fortunately, the fact that the standing wave system is of long period permits a simplification of Eq. 16. Thus, provided $\frac{m d}{2 B} < \frac{1}{20}$ we may introduce the approximations $\sinh k d \simeq k d$, $\sinh k s \simeq k s$, $\cosh k d \simeq 1$. For B = 2100 ft and $d \simeq 45$ ft, this is valid to at least the 5th mode. Thus, noting from Eqs. 1 and 17 that $\sigma \simeq k \sqrt{g d}$,

$$\zeta = A \sqrt{\frac{g}{d}} \; \frac{\sin k l}{k l} \; \sin k b \qquad \text{(18)}$$

From Eq. 18 it is seen that the excitation must vanish when either sin k l or sin k b is zero. Thus, the ship will tend to have no motion if

$$\left.\begin{aligned} l &= \frac{B}{m} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19a) \\ b &= \frac{B}{m} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19b) \end{aligned}\right\}$$

This implies that the ship will lie quietly if either its half-length l or its distance b from the end boundary is an integral submultiple of the width of the dock B. On applying the test we find $B/l = \frac{2100}{240} = 8.76$ and $B/b = \frac{2100}{590} = 3.56$ and conclude that the ship was inevitably prone to surging.

The circumstances in which the ship would be liable to maximum excitation would be such as to make either sin k l or sin k b unity, for which

$$\left.\begin{aligned} l &= \frac{p\,B}{2\,m} \\ b &= \frac{p\,B}{2\,m} \end{aligned}\right\} \quad p = (1, 3, 5\ldots) \quad \ldots\ldots\ldots\ldots \quad (20)$$

For the various modes of oscillation shown in Fig. 2 ($m = 1$ to 7), the relative magnitudes of the amplitude factor ξ/A of the forcing function are shown in Table 4. It is seen that the uninodal, binodal, quinquinodal and possibly sextinodal seiches emerge as having greatest capacity for exciting the surge motion of the ship, a fact that may be inferred from Fig. 2, because for these modes of oscillation the ship center lies closest to the nodes where horizontal velocities of flow are highest.

TABLE 4.—COMPARATIVE MAGNITUDES OF MODES OF EXCITATION

Mode or Nodality of Seiche m (1)	Wave Number (2)	Ship-Position Relationships			$(\delta/A)(\sec^{-1})$	
		$\frac{\sin kl}{kl}$ (3)	sin kb (4)	$\frac{\sin kl}{kl}$ sin kb (5)	Low Tide (6)	High Tide (7)
1	1.50	0.98	0.77	0.76	0.65	0.63
2	2.99	0.95	0.98	0.93	0.79	0.78
3	4.49	0.90	0.47	0.43	0.36	0.36
4	5.99	0.83	0.38	0.32	0.27	0.26
5	7.48	0.74	0.96	0.71	0.60	0.59
6	8.98	0.64	0.83	0.53	0.45	0.44
7	10.48	0.53	0.10	0.05	0.05	0.04

The solution to Eq. 12, representing the forced oscillation, may be stated[4,10] as

$$X = X_1 \cos(\phi - \alpha) \quad \ldots\ldots\ldots\ldots\ldots \quad (21)$$

in which, for the case of negligible damping ($2\beta \rightarrow 0$) - as would apply to the case being considered - the maximum amplitude of surge X_1 and the phase angle α are defined by

$$X_1^n \, \Delta(n) - \left(\frac{\sigma}{\omega}\right)^2 X_1 = - \frac{\zeta}{\omega}\left(\frac{\sigma}{\omega}\right) \quad \ldots\ldots\ldots\ldots (22a)$$

$$\tan \alpha = \frac{\sigma}{2\beta\omega} \left[\frac{(2\beta)^2}{X_1^{n-1} \, \Delta(n) - \left(\frac{\sigma}{\omega}\right)^2} - 1 \right] \quad \ldots\ldots (22b)$$

In Eqs. 22, $\Delta(n)$ is a numerical function of n only, whose values have been given elsewhere.[4,10] For present purposes it is sufficient to state that the values of $\Delta(n)$ applicable to the n-values found in Table 3 are as will be given subsequently in Table 5.

TABLE 5.—PARAMETERS IN THE EQUATION OF SURGE MOTION

Time on May 5, 1952 (1)	Tidal Condition (2)	Water Depth d, in feet (3)	Rope Constants			Equivt. Natural Frequency ω, Eq. 13b[b] (7)	$\frac{\zeta}{\omega A'}$ Eq. 23[c] (8)
			n (4)	C[a] (5)	Δ(n) (6)		
0700	Low	44	3.3	10.0	0.725	0.127	6.75
1200	High	46	2.4	26.1	0.805	0.200	4.17

[a] tons ft^{-n} [b] $\text{ft}^{\frac{1-n}{2}} \text{sec}^{-1}$ [c] $\text{ft}^{\frac{n-1}{2}}$

From Eqs. 13a, 13b and 18 the value of $\frac{\zeta}{\omega}$ may be written

$$\frac{\zeta}{\omega} = A' \sqrt{\frac{C_M W}{C d}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

in which

$$A' = A \frac{\sin k l}{k l} \sin k b \quad \ldots\ldots\ldots\ldots (24a)$$

$$C_M = \frac{M_x'}{M} = \left[1 + \left(\frac{M_x''}{M}\right) \right] \quad \ldots\ldots\ldots\ldots (24b)$$

and

$$M_x'' = C_x \frac{\pi}{8} \rho B^2 D \quad \ldots\ldots\ldots\ldots\ldots\ldots (24c)$$

In these definitions A' is the effective seiche amplitude, W the dead weight of the ship, C_x an inertia coefficient whose value depends on the ship beam-length ratio (a/2l), and ρ the mass density of water. The block-length of the ship required to displace 18,000 tons of water is $2l = 350$ ft and the beam-length ratio is thus $a/2l = 0.19$, for which C_x in Eq. 24c, is about 2.0.[10] For the dimensions of the ship, then, M_x'' is found to be 1.99×10^5 and $M = 12.52 \times 10^5$ slugs. Thus, $C_M = 1.16$, and the forcing function term $\left(\frac{\zeta}{\omega}\right)$ of Eq. 23 assumes the values listed in Table 5 for the values of C and d given in Table 3.

In solving Eq. 22a it is convenient to write $\xi = \sigma/\omega$ and derive the roots of the quadratic in ξ in terms of A' and X_1. Then, by choosing different values of A' and X_1, the families of curves shown in Fig. 6 are obtained. The interrelated quantities ξ, A' and X_1 are plotted as X_1 versus τ with isolines of A', τ being derived directly from $\sigma = (2\pi/\tau) = \omega\,\xi$.

The undamped free oscillation, for which the excitation ζ in Eq. 22a is zero, is amenable to exact solution[4,10] and yields the 'backbone' curves of Figs. 6(a) and 6(b).

RESPONSE OF THE SHIP TO THE VARIOUS MODES OF EXCITATION

It was shown in Table 4 that the seiches most likely to influence the ship and promote surging in virtue of the ship's length and position in the dock would be the fundamental transverse oscillation in the dock and its second and fifth harmonics (Fig. 2). Vertical ordinates drawn in Figs. 6 at the values of τ applicable to these modes (Table 1) define the behavior-patterns of the ship at these individual frequencies of excitation. As an example, consider a binodal seiche $\tau = 55.7$ sec prevailing at low tide, Fig. 6(a). Assuming that the effective amplitude A' were to increase from zero at a reasonably slow rate in time, the ordinate tells us that at the amplitude of $A' \simeq 0.065$ ft the amplitude of ship movement would be $X_1 \simeq 0.58$ ft. Then suddenly, if A' increased further, the ship movement would jump through an instability to 1.2 ft (on the upper side of the 'backbone' curve). This trend is illustrated in another way in Fig. 9, wherein the further increase of seiche amplitude A' at low tide is shown to follow the uppermost curve. The corresponding curve obtained from the ordinate at $\tau = 54.5$ sec at high tide [Fig. 6(b)] forms the lowest curve of Fig. 9. Interpolated between are curves such as might apply at hourly intervals between low and high tides.

Figs. 8 and 10 correspond to the conditions of a uninodal seiche and quinquinodal seiche respectively and are derived similarly from Figs. 6. The curves of Fig. 10 are rather notably different from Figs. 8 and 9 because of the larger jump tendencies to which the ship is subject as a consequence of the non-linear instability.

From Fig. 7(b), the amplitudes A of the uninodal, binodal and quinquinodal seiches, prevailing on the hour, have been recorded and tabulated in Table 6. The effective seiche amplitudes, A', derived directly from Eq. 24a are also given from information provided in Table 4. The values of A' given in columns (6), (7) and (8) have been plotted in Figs. 8, 9 and 10 respectively to yield the circled points on the hourly lines of X_1 versus A'. The connecting heavy lines define the continuous response of the ship in surge to the increasing amplitudes A' that occurred with the rise of tide.

It is seen from Fig. 8 that the amplitude of surge movement resulting from the uninodal seiche alone would not have exceeded about 0.95 ft throughout the tide rise. From Fig. 9 it is also clear that the second harmonic seiche of it-

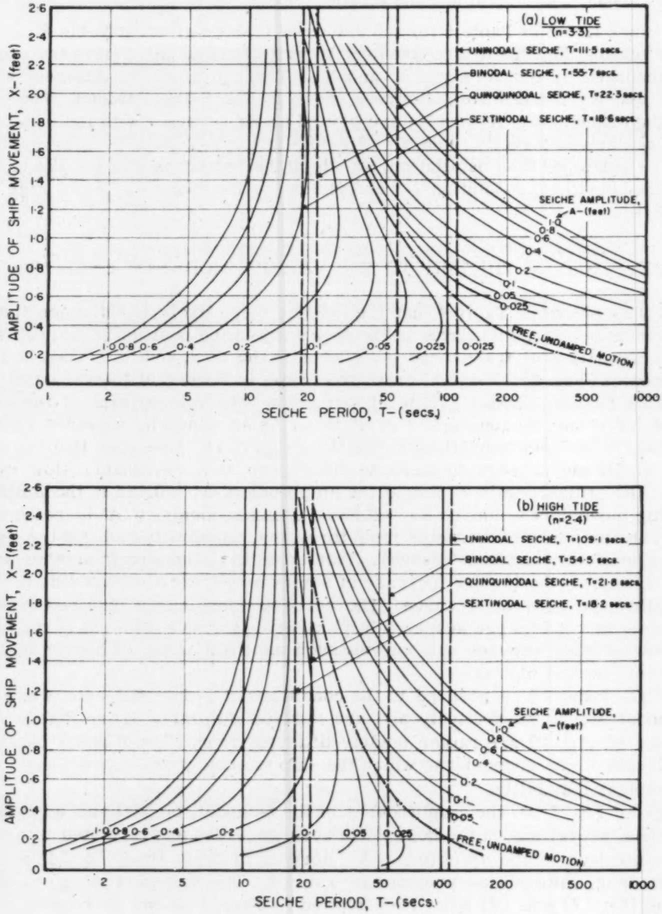


FIG. 6.—CHARACTERISTICS OF NON-LINEAR SURGE MOTION OF SHIP

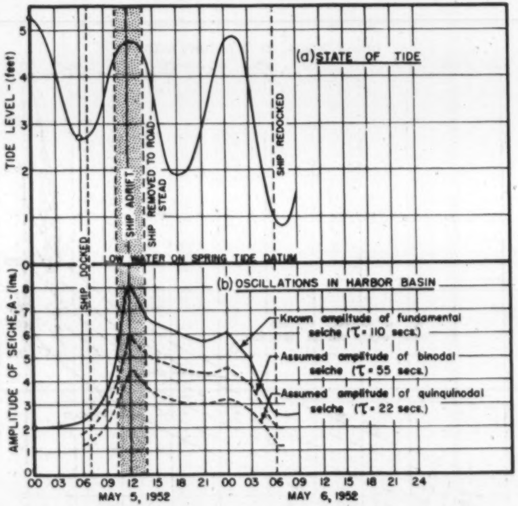


FIG. 7.—TIDE AND SURGE CONDITIONS PREVAILING IN THE HARBOR BASIN

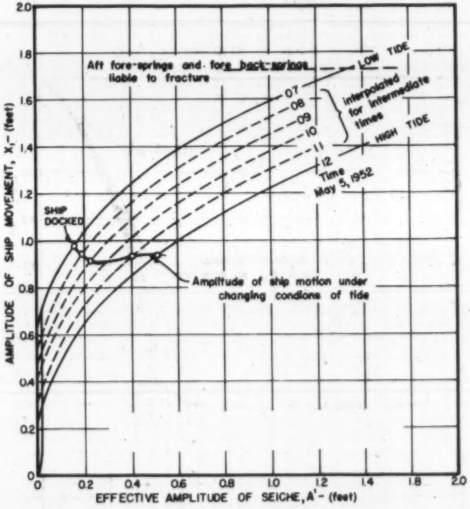


FIG. 8.—CONTINUOUS SURGE RESPONSE OF OIL TANKER TO UNINODAL TRANSVERSE SEICHE OF 110 SEC PERIOD

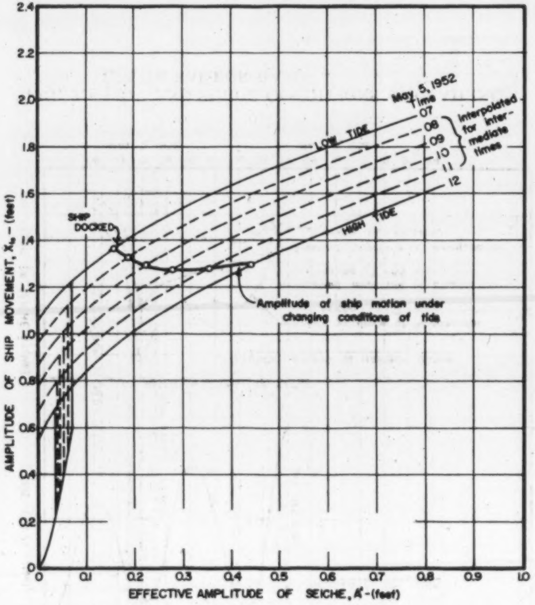


FIG. 9.—CONTINUOUS SURGE RESPONSE OF OIL TANKER TO BI-NODAL TRANSVERSE SEICHE OF 55 SEC PERIOD

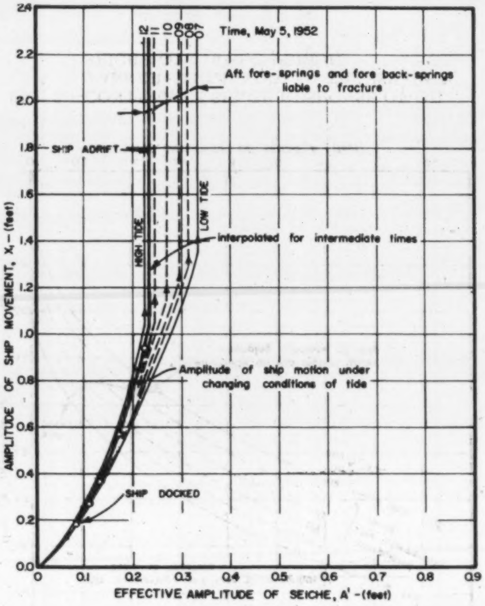


FIG. 10.—CONTINUOUS SURGE RESPONSE OF OIL TANKER TO QUINQUINODAL TRANSVERSE SEICHE OF 22 SEC PERIOD

self would have been incapable of causing a larger surge movement of the ship than $X_1 \simeq 1.3$ ft. In the case of the quinquinodal sieche, however, Fig. 10 shows that the ship response to amplitude growth would have been progressive and that at some time between 1100 and 1200 the ship surging would have reached critical instability and caused over-loading and probable failure of the fore- and backsprings. Once these mooring lines had ruptured, the remaining mooring lines would be rapidly overloaded and could be expected to follow suit.

This analysis then would seem to indicate that the existence of a 5th harmonic seiche of 22 sec period, of only 3 to 4 in. amplitude, could have resulted in the breaking adrift of the ship. According to the evidence the ship was in serious difficulties at least one half-hour earlier than the analysis suggests, which could mean that the amplitudes of the quinquinodal seiche were actually larger than assumed in Fig. 7(b), or that the combined effects of several seiches were at work.

TABLE 6.—PROBABLE REAL AND EFFECTIVE AMPLITUDES OF SEICHES

Time on May 5, 1952	Tidal Condition	Probable Real Ampli, A in inches			Effective Ampli, A' in inches		
		m = 1	m = 2	m = 5	m = 1	m = 2	m = 5
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0700	low	2.6	2.0	1.4	2.0	1.9	1.0
0800		3.0	2.4	1.8	2.3	2.2	1.3
0900		3.6	2.9	2.2	2.7	2.7	1.5
1000		4.6	3.6	2.8	3.5	3.3	2.0
1100		6.5	4.7	3.7	4.9	4.4	2.6
1200	high	8.0	6.0	4.4	6.1	5.6	3.1

The effect of a sextinodal seiche of 18 sec period has not been investigated on grounds that its effective amplitude, A', would have been only 0.53 (Table 4) of its probable real amplitude, A. The probable real amplitude might be expected to have been less than that of the 5th harmonic. If this were not the case and the amplitude A of the 6th harmonic were large, the sextinodal seiche could also have been a contributing factor with results similar to Fig. 10.

SUMMARY AND CONCLUSION

The results of this study show that the 18,000-ton oil tanker, in virtue of her berthing location, her length and the characteristics of her mooring ropes, was innately responsive to longitudinal surge motion excited by the fundamental transverse seiche in the dock and its second, fifth and possibly sixth harmonics. Indications are that the first two of these oscillations, despite their comparatively large amplitude growth on the rising tide, would not have stimulated any very serious longitudinal motion of the ship; in fact the motion remained indifferent to the increasing magnitude of these sea disturbances. On the other hand the fifth harmonic transverse seiche, even with quite modest amplitude growth (up to 4 in.), would have excited an increasing surge movement of the ship up to the point that the ship would suddenly and violently have lunged fore and aft and thereby strained the 18 in. coir springs of the mooring lines beyond their limits of recovery. It must be supposed that the co-existence of these several modes of sea oscillation could have greatly aggravated and hastened the failure of the mooring lines.

The reasons for the ship's critical response to these lesser and higher-frequency disturbances must be sought in the peculiarities of the characteristic curves (Figs. 6), relating seiche amplitude A' and period τ with surge amplitude X_1, that form the solution of the non-linear equation of longitudinal ship motion. Obviously of great importance is the resiliency of the mooring lines as reflected in the values of C and n. These values largely dictate how the "backbone" curve will be disposed in relation to the exciting periods and how steeply it will rise with increase of X_1. Clearly, if the "backbone" curves of Fig. 6 were to rise less sharply the conditions could be considerably improved to the extent that at the period of the fifth harmonic (22 sec) the instability jump might conceivably be contained within values of X_1 that would not overstrain the ropes. The possibility that still higher frequency excitations would then become critical could be ruled out on the basis that there are inherent tendencies towards evanescence of increasingly higher harmonics, quite apart from the decay features of the factor $(\sin kl/kl)$ with increasing nodality m, (exhibited in column (3) of Table 4), that tend to reduce the effective seiche amplitude A'.

It would seem that in harbors subject to surging much might be done towards alleviating dangerous conditions for ships of certain sizes at certain berths by judicious design of the mooring systems to be used in given circumstances.

ACKNOWLEDGMENT

Submission of this paper is made possible with the assistance, in part, of the Office of Naval Research (David Taylor Model Basin), Contract No. N onr-2119 (02), that is gratefully acknowledged.

APPENDIX.—NOTATION

A = Amplitude of vertical surface elevation of seiche;

A' = Effective amplitude of seiche, Eq. 24;

a = Beam or breadth of ship;

B = Width of dock or basin;

b = Distance of center of ship from one end of dock;

C = Non-linear spring constant;

C_M = Coefficient of virtual mass, Eq. 24 b;

C_x = Inertia coefficient in surge, Eq. 24 c;

D = Draft or wetted depth of ship;

d = Water depth;

g = Acceleration due to gravity;

H = Height of rope fairleads on ship above quay level;

k = Wave number $\left(= \frac{m\pi}{B}\right)$;

L = Horizontal projection of stretched mooring line;

L_o = Horizontal projection of mooring line for stationary ship in equilibrium position;

dL = $\left(L - L_o\right)$;

l = Half-length of ship represented as a rectangular block $(= M/2\,\rho\,BD)$;

M = Mass of ship;

M'_x = Virtual mass of ship for surge motion in x-direction $\left(= M + M''_x\right)$;

M''_x = Added hydrodynamic mass for surge motion in x-direction;

m = Integer $(1, 2, 3 \ldots)$ defining nodality or harmonic mode of seiche;

N_x = Linear damping coefficient in surge motion;

n = Numerical exponent depending in value on rope characteristics;

p = Odd integer $(1, 3, 5 \ldots)$;

R_x = Total restoring force in x-direction from all mooring lines acting bowwards (or sternwards);

S = Length of stretched mooring line;

S_o = Length of mooring line for stationary ship in equilibrium position;

ΔS_o = Excess of S_o over $\sqrt{L^2 + H^2}$;

dS = Stretch of mooring line under tension $\left(= S - S_o\right)$;

s = Clearance between sea bed and ship keel $(= d - D)$;

T = Tension in mooring line at the ship;

T_h = Horizontal component of T;

T_x = Component of T_h in x-direction;

t = Variable time;

W = Dead weight of ship;

X = Variable horizontal longitudinal (surge) translation of mass-center of ship from rest position;

X_1 = Amplitude of forced oscillation of ship in surge;

$\dot{X}, \ddot{X}$ = Variable horizontal surge velocity and acceleration of mass-center of ship;

x = Variable horizontal distance from origin in still water surface at one long side of dock, transverse thereto;

Y_0 = Horizontal distance of center-line of ship from quay wall;

α = Phase angle, Eqs. 21 and 22;

β = Damping factor, Eq. 13a;

$\Delta(n)$ = Function of n only;

ϵ = Phase angle;

ζ = Excitation factor, Eqs. 12 and 16;

η = Variable vertical elevation of water surface, with reference to still water level;

θ = Angle between horizontal projection of mooring line and side of quay (Fig. 3);

ξ = Ratio σ/ω;

π = Universal constant (3.14159);

ρ = Mass density of water;

σ = Angular frequency of seiche ($= 2\pi/\tau$);

Σ = Summation symbol;

τ = Period of seiche, Eq. 1;

ϕ = Angle ($= \sigma t + \epsilon$); and

ω = Equivalent natural frequency of non-linear surge oscillation of ship, Eq. 13 b.

DISCUSSION

W. C. Q. JOOSTING.[12]—The author is to be commended for having made a comprehensive application of surge theory, as far as developed to date, to the case of a particular ship. He arrives finally at the conclusion that the fifth harmonic (period 22 sec) of the known fundamental seiche across the harbour basin in question must have been responsible for the parting of the ropes of an 18,000 ton tanker.

The writer has been intimately connected with this occasion and would like to put forward some remarks on the details of Wilson's theory and some of the assumptions made, in the light of the more detailed knowledge available locally as regards the conditions in the harbour in question. As a result, he also wishes to present a different possible explanation of the cause of the ship breaking loose; in the writer's view, it is quite likely that the excessive movement of the ship was caused by the combined action of some 7 seiches or 8 seiches with periods in the band of about 35 sec to 120 sec, which two different models have shown to exist in that part of the harbor.

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[12] Dist. Engr., Research, South African Railways.

Author's Theory and Application.—The general theory of the surging of a ship moored at a quay, along which a seiche is active, is now well established and explains clearly the principles involved in the motion of the ship, and the author has presented a very interesting and able application of this theory. However, there are some points of major and minor importance which call for clarification. It is hoped that the author will not take amiss the rather large number of questions raised. This is done to bring the theory nearer to its fully correct solution. The points will be treated in the order in which they appear in the paper.

a. In Table 1 the author gives a list of the harmonics of the transverse seiche in the harbour basin. On the tide gauge in that basin, only the fundamental period of about 110 sec is clearly visible, due to the small time scale and throttling of the water duct. This seiche is already not pure as it contains two nodes, one along and one across the middle of the rectangular basin. In his previous work, the author[13] had already found, in addition, another strong seiche, at about 99 sec (uncorrected), which was found to have one node along but two across the basin, and he, also, gave a formula with which the periods of seiches with various numbers of nodes in either direction could be calculated. Later model work by the writer, and confirmed by similar tests by a French firm, has shown that the main combinations given in Table 1 exist in this basin.

TABLE 7.—MAIN RESONANT SEICHES IN HARBOUR BASIN

Number of nodes												
	along	1	1	1	1	2	2	2	3	3	4	5
	across	1	2	3	4	1	2	4	3	4-5	4-5	5-6
Period, in seconds		106	94	80	69	59	54	49	36	34	27	21

Thus, the seiches active in this basin are more numerous and of different constitution than those assumed to exist by the author.

b. The author states that he has conjectured the amplitudes of the various harmonic waves as shown in Fig. 7(b). Here two matters arise:

The amplitudes of the fundamental 110 sec seiche appear to be those indicated by the tide gauge. However, this gauge is located at a distance of several hundred feet from the corner of the basin, the wave height at which would be a measure of the seiches at the berth in question. Now, the model tests have shown that the wave height in that corner is about 40% greater than that at the tide gauge position, in respect of the 110 sec seiche. Accordingly, the wave height used in the calculations should be taken larger by that amount.

On the other hand, it would seem to the writer that the wave heights assumed for the harmonic seiches are a little on the high side. The author gives, in Fig. 7(b), a ratio of the fundamental and second and fifth harmonics of about 1 : 0.75 : 0.55. It is not stated on what basis these heights were adopted; the South African and French model tests show that operation with a constant input wave height at the wave generator results, due to the layout of the harbour, its protection by the breakwater and the position of the entrance to the basin, in approximate direct proportionality between the heights of the main resonance peaks and the wave period. Consequently, it is thought that a ratio of 1 : 0.5 : 0.2 would have been more appropriate as a likely assumption.

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[13] "Origin and Effects of Long Period Waves in Ports," by B. W. Wilson, XIXth Internatl. Navigation Congress, 1957, Sect. II, Communication 1, Chap. 11, p. 35, Fig. 13.

But in that case, the fifth harmonic would become too small to be responsible for excessive movement of the ship so that the author's conclusion would be invalidated.

c. In the same figure, it is seen that at the time of docking, due to the assumptions made in respect of the relative slacks at the three mooring positions, the back-springs (4) take almost all the strain consequent on movement of the ship. Would not the ropes be made fast in such a way that they would be subject to approximately equal stresses at the relatively small movement of the ship at docking? This would reverse the author's assumptions as to the relative slacks: (1) and (2) would have the least and (4) the greatest slack instead of vice-versa; all three curves would intersect at a point of equal stress, say 4 tons, at a ship deflection of, say, 1.3 ft.

d. In the development of the equations of motion (Eq. 12), the author uses the mathematical contraction ζ which has the dimension of a velocity and, as such, has little meaning. A real convenient physical quantity would have been obtained by using the factor $\zeta/\sigma = -H'$. In that case, Eq. 16 would become, with $\sigma \simeq \sqrt{gd}$:

$$-\frac{\zeta}{\sigma} = H' = H\,\frac{\sinh kd - \sinh ks}{kD \cosh kd}\;\frac{\sin k\,l}{k\,l}\;\sin kb \quad . \; . \; . \; . \; . \qquad (25)$$

in which H is the amplitude of the horizontal water movement at the node of the seiche, and H', consequently, denotes the actual horizontal "driving" amplitude, corrected for depth of immersion of the ship, its length and its position in the seiche.

e. The writer considers that, to be fully correct, another correction factor should be added to Eq. 25 in order to account for the fact that the seiche is disturbed by the proximity of the ship to the quay and the seabed; however, no suitable values seem to be available as yet for this factor which can only be determined experimentally.

f. As regards the influence of the ship's length, the author's factor $\frac{\sin k\,l}{k\,l}$ relates to the influence of the acceleration of the water past the ship only. But this influence is small ($C_M - 1 \simeq 0.16$) as compared with that of the slope of the wave on which the ship rides down, which is the main driving force. With a ship as long as the wave ($k\,l = \pi$), this factor would become zero, which is obviously impossible as, with such a ship at the node of the wave, one half of the ship is pushed upwards while the other half is pulled down so that it cannot lie in the horizontal plane. This is shown clearly in Fig. 11.

The horizontal position is only reached with a ship that is considerably longer than the seiche wave. The full formula was published elsewhere[14] by the writer in 1957. However, the acceleration of the water had then been taken at full value ($C_M = 2$) whereas, later developments have shown that it only acts to the extent of ($C_M - 1 \simeq 0.16$). The correct factor for the influence of the length of the ship is, therefore,

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[14] "Investigation into Long Period Waves in Ports," by W. C. Q. Joosting; XIXth Internatl. Navigation Congress, 1957, Sec. II, Communication 1, Eqs. 17 and 21, p. 225.

$$c_1 = \frac{\left(C_M - 1\right)\frac{\sin k\,l}{k\,l} + 3\left[\frac{\sin k\,l - k\,l\cos k\,l}{(k\,l)^3}\right]}{C_M} \quad \ldots\ldots(26)$$

The length 2l of the ship used in the calculations must, of course, be the "effective" length which is slightly less than the overall length due to the shape of stern and bow. The difference will be of the order of 10% to 15% so that in this case an effective length of about 420 ft should be reasonable. Using this length with a ship as long as the wave and the author's value $C_M = 1.16$, the factor becomes $c_1 = 0.262$ which is anything but zero. Similarly, the horizontal position of the ship is obtained when the effective length of the ship is 1.362 times that of the wave.

This factor is represented in Fig. 12, as a function of the ratio between the effective length of the ship and that of the wave, with $C_M = 1.38$, which includes the effect of the proximity to quay and seabed and would seem to cover most cases well.

h. The author puts forward Eq. 19 to prove that the ship was inevitably prone to surging. In Eq. 19(a), apart from the fact that this condition would have to be revised according to the writer's Eq. 26; the author uses the overall length of the ship. Had he used the more correct effective length he would have obtained $\frac{B}{l} = \frac{2100}{210} = 10.0$, meaning that the 10th harmonic wave (period 11 sec) would not contribute to surging of the ship. In the writer's opinion the statement should rather be that "ships are inevitably prone to surging, except at those periods for which Eq. 19 give a value in the vicinity of an integer number."

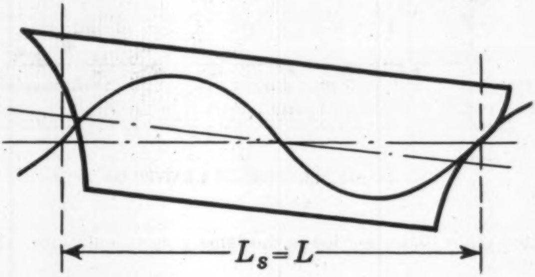

FIG. 11.—SLOPE OF SHIP AT NODE OF SEICHE

i. Eq. 20(a) would have to be corrected to fit the writer's Eq. 26. Values of p greater than unity need hardly be considered as then k l becomes large so that $\frac{\sin kl}{kl}$ becomes small with, consequently, little action on the ship.

j. It is not clear how the author obtained a cosine function in Eq. 21. The original differential equation leads naturally to

$$X = X_1 \sin(\phi - \alpha) \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

The cosine function leads to an unnatural phase angle which has to be corrected by 90° to get the correct relationship between the ship movement and that of the water.

k. Had the author used, in Eq. 22(a), the writer's factor $-\frac{\zeta}{\sigma} = H'$, this equation would have reduced to

$$X_1^n \ \Delta\,(n) \ \left(\frac{\omega}{\sigma}\right)^2 - X_1 = H' \quad \ldots\ldots\ldots\ldots\ldots \quad (28)$$

which would then have constituted a physically clear relationship between the properties of the ship, the moorings and the water.

l. Eq. 22, apart from the prior-mentioned shift by 90°, seems to contain an error; as given, the phase angle is permanently tan $\alpha = -\infty$, when the damping

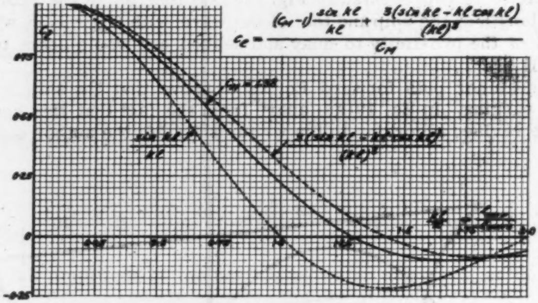

FIG. 12.—INFLUENCE OF LENGTH OF SHIP

is neglected $(2\beta \rightarrow 0)$, as is done in the remaining calculations. The expression should read

$$\tan\,\alpha = \frac{\sigma}{2\beta\ \omega}\left[\frac{(2\beta)^2 + \left(\frac{\sigma}{\omega}\right)^2}{X_1^{\,n-1}\ \Delta\,(n)} - 1\right] \quad \ldots\ldots\ldots\ldots \quad (29)$$

However, with the sine function (Eq. 27), the correct phase angle—that between the horizontal movements of the ship and water—is given by

$$\tan\alpha = \frac{\left(\frac{2\beta\ \omega}{\sigma}\right)\ X_1^{n-1}\ \ \Delta\,(n)}{(2\beta)^2 + \left(\frac{\sigma}{\omega}\right)^2 - X_1^{\,n-1}\ \ \Delta\,(n)} \quad \ldots\ldots\ldots\ldots \quad (30)$$

This is in agreement with the phase angles for normal linear spring systems.

m. From Fig. 7(a), the amplitude of the 110 sec seiche, at the time of maximum response at noon, was 8 in. At the corner of the basin, as explained above, its amplitude would be 1.4 x 8 = 11.2 in. The fifth harmonic, as stated by the writer, would possibly have an amplitude at that position of $\frac{11.2}{5 \times 12} = 0.187$

ft. If, further, this wave amplitude is corrected for the position of the ship and its length, the effective wave amplitude will be $A' = 0.187 \times 0.96 \times 0.755 = 0.136$ ft. Applying this value to the response curves of Fig. 9, it is seen that at no time could the ship have been surging excessively due to the effects of the fifth harmonic of the seiche unless the sea wave exciting it was about twice as high as the fundamental 110 sec wave, which seems highly unlikely for various reasons.

Alternative Solution to Problem.—In view of the foregoing, supported by the writer's experience that ship surging, generally, does not seem to contain much movement at periods of less than about 30 sec, the problem has been examined again and it has been found that, in this particular case, it may well be that the ship broke loose due to the combined action of several of the longer period water oscillations listed in Table 7.

Of course, once a few ropes had parted, the maximum response of the ship shifted to slightly larger periods with an increase in the maximum tension per rope, so that then only one or two periods, probably between 30 sec and 40 sec, may have been responsible for the inability to re-moor the ship.

For clarity, the basic response Eq. 29 is given here once more, using the various physical factors involved

$$\frac{R_x}{c_n\, c_b\, C_M\, M} - \sigma^2\, X_1 = H' = -\, c_l\, c_p\, \sigma\, A \sqrt{\frac{g}{d}} \quad . \; . \; . \; . \quad (31)$$

where, in addition to the author's notation c_n denotes the factor for exponential response $= 1/\Delta(n)$, c_b is the factor for proximity to quay and seabed $\simeq 1.2$, c_l is the factor for length of ship = Eq. 26, c_p represents the factor for position of ship $= \sin kb$, A is the amplitude of seiche in corner of basin, and H' denotes the effective amplitude of horizontal water movement.

Strictly speaking, the proximity of the ship to quay and seabed will have a different influence on the water acceleration and wave slope, respectively, but no separate values are available. It would seem that $c_b = 1.2$ could be accepted as a reasonable approximation.

Furthermore, as previously mentioned, it was found that, in the basin in question, the amplitudes of the resonant seiches could be considered to be proportional to their periods so that the quantity σ A can be replaced by the factor $2\pi c_a A_o/\tau_o$, where A_o is the amplitude of the 106 sec seiche discernible on the tide gauge, τ_o its period and $c_a = 1.4$ to account for the difference in height of the waves indicated on the gauge and in the corner of the basin. This gives $\sigma A = 0.083\, A_o$.

The value of $\Delta(n)$ can be obtained from a rather cumbersome integration. However, the factor $c_n = \frac{1}{\Delta(n)}$ can be very closely represented by $c_n = 1 + 0.2\,(n - 1)$.

The values of the other quantities are, as given by the author: $C_M = 1.16$; d = 43 ft; g = 32.14 ft per sq sec at the latitude of the harbour, and M = 560 tons sq sec per ft. The values c_l and c_p have to be obtained from their respective equations for each seiche.

The solution will be developed for the conditions at the time the ropes began parting, that is 10:30 a.m. From the author's Figs. 4 and 5 the mooring force—

ship amplitude relation—can be assessed at approximately:

$$R_x = C \, X_1^n = 18.5 \, X_1^{2.8} \quad \ldots\ldots\ldots\ldots\ldots \quad (32)$$

Inserting the Eq. 32 into Eq. 31 and using only mooring tensions results in

$$R_x - \frac{14763 \, R_x^{1/2.8}}{\tau^2} = - \, 76.38 \, A_e \quad \ldots\ldots\ldots\ldots \quad (33)$$

in which $A_e = c \, c_p \, A_o$ is now the effective water amplitude indicated by the tide gauge in respect of the fundamental 106 sec seiche, corrected for length and position of the ship.

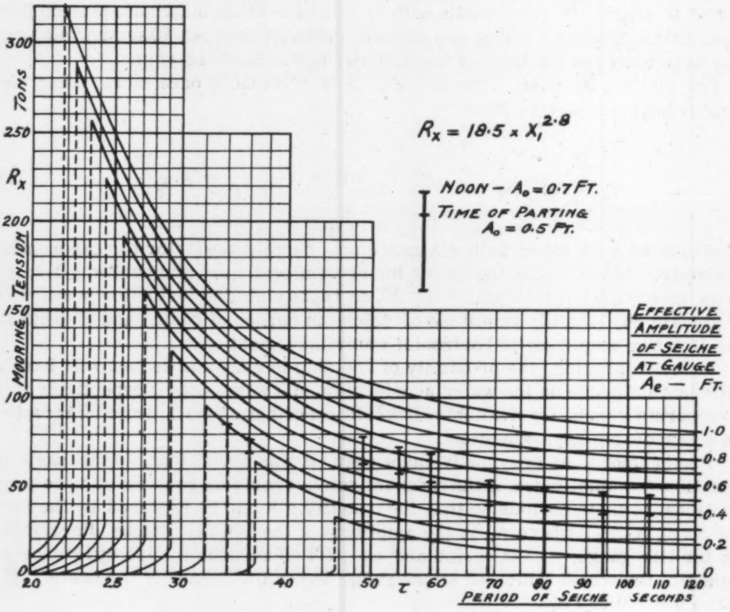

FIG. 13.—SHIP RESPONSE CURVES

Fig. 13 shows the relationship between the mooring forces R_x and the period τ of the seiche at various effective seiche amplitudes A_e from 0 ft to 1 ft.

Now, the effective water amplitude A_e can be found from the fact that, at the time of parting of the ropes, the fundamental seiche indicated on the tide gauge had an amplitude of about $A_o = 0.5$ ft, as shown in Fig. 7(b). The corrections to be applied for the various seiches are listed in Table 8 as well as the resulting effective amplitudes A_e.

Plotting these values of A_e as vertical lines in Fig. 13 at their corresponding periods gives the various mooring forces which would be obtained if each seiche were acting alone. It is seen that only the longer periods cause relatively large forces; the short periods lie below the maximum response conditions.

If the forces, due to the various seiches, are added together, a total force of about 390 tons is obtained. However, it must be considered that all the component movements are hardly ever in phase with each other. It would seem reasonable to expect that surges on the moorings of about half this maximum, or 195 tons, can be expected fairly frequently at the appropriate moments. Now, because there are the equivalent of 12 coir ropes of 18 in. holding the ship in each direction, the average surge force per rope would be about 16 tons.

Considering the influence of age, distribution and unequal response of the moorings, a mean breaking strength of 15 tons can be taken as reasonable, as stated by the author under the heading "Elastic Properties of the Mooring Ropes," so that it seems quite possible that parting of the ropes was initiated in this manner.

TABLE 8.—EFFECTIVE SEICHE AMPLITUDES

Period, in Seconds	Nodality		c_1	c_p	A_o, in feet	A_e, in feet
	m	n				
106, 94, 80, 69	1	1,2,3,4	0.988	0.773	0.5	0.381
59, 54, 49	2	1,2,4	0.955	0.981	0.5	0.468
36, 34	3	3, 4-5	0.900	0.473	0.5	0.213
27	4	4-5	0.825	0.381	0.5	0.157
21	5	5-6	0.737	0.956	0.5	0.352

Subsequently, surging increased to reach a maximum, at noon, of $A_o = 0.7$ ft. The mooring forces increased correspondingly, and an additional force, at a period of 34 sec, overcame the jump-period, so that at noon, if all the moorings had still been present, the 50% total force would have reached 288 tons, or 24 tons per rope. In addition, the moving liquid load of the tanker may have assisted the surging.

As a matter of interest Fig. 14 has been compiled, giving the response conditions with the same basic values except that the moorings are assumed to have 1 ft more slack, so that $R_x = 6.9\ X_1^{3.5}$ represents the approximate mooring characteristics.

It is seen that the periods of maximum response now lie at somewhat higher values, and that the mooring forces at the periods beyond the maximum response are a little greater, while the maximum response forces have decreased a little. The effect of the shift in the maximum response periods is that the 34 sec and 36 sec periods do not contribute any more. At the same time, the movement at the longer periods leads to slightly higher mooring forces. The net result is that the 50% total force is about the same as before—192 tons or 16 tons per rope—at the time of parting.

At noon, the time of maximum surge conditions, the water amplitudes were still not high enough to cause any shorter periods to have any influence; the total 50% force then became 245 tons or 20 tons per rope which is some improvement on the 24 tons with tauter moorings. Under certain conditions, it may, therefore, be of advantage to slacken the ropes.

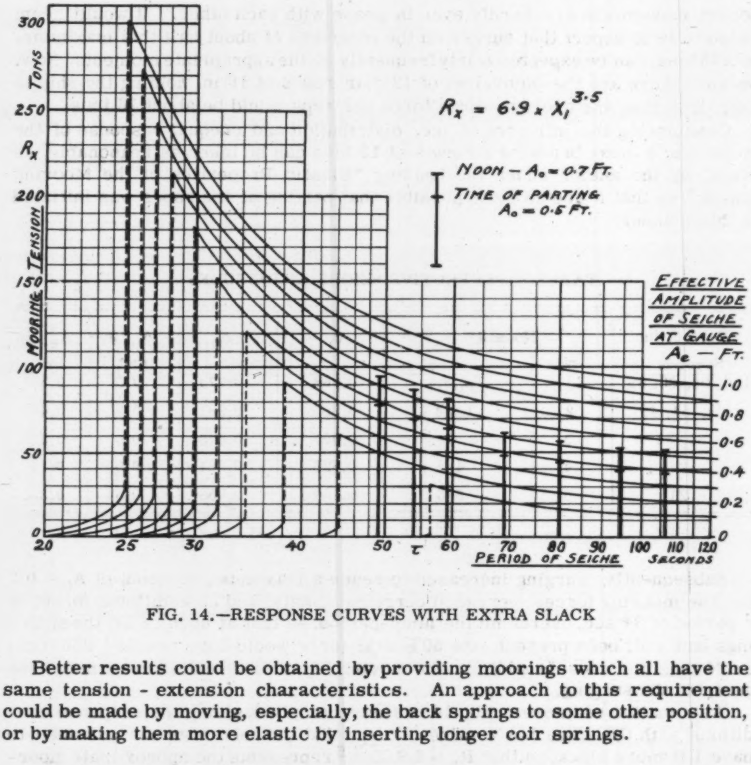

FIG. 14.—RESPONSE CURVES WITH EXTRA SLACK

Better results could be obtained by providing moorings which all have the same tension - extension characteristics. An approach to this requirement could be made by moving, especially, the back springs to some other position, or by making them more elastic by inserting longer coir springs.

BASIL W. WILSON,[15] **F. ASCE.**—In replying to the discussion it seems appropriate to deal with the points as brought out by Joosting.

(a) The rather simplified representation of the transverse seiches existing at the end of the long rectangular basin, given in the author's Fig. 2 and Table I, is queried by Joosting. It is indeed correct that the two dimensional character of the rectangular dock has an important influence on the periods and properties of the possible model of oscillation that can exist. Eq. 1 should properly be replaced by

$$\tau_{m,n} = \frac{2\,B}{\sqrt{g\,d}} \left[\frac{1}{m^2 + \gamma^2\,n^2}\right]^{\frac{1}{2}} \quad \ldots\ldots\ldots \quad (34)$$

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[15] Prof., Engrg. Oceanography, A & M College of Texas, College Station, Tex.

in which $\gamma = B/\lambda$ is the breadth-length ratio for the dock, while m and n(= 0, 1, 2, 3, . . .) are integers defining the co-existing harmonic modes or number of nodes in the transverse and longitudinal directions respectively of the rectangular basin. As mentioned by Joosting the author had indeed presented this formula in the cited paper,[2] although he had actually given it first in a still earlier paper[16] in which he had tabulated values of the periods $\tau_{m,n}$ for different values of m and n. These agreed generally with the values given by Joosting in his Table I except that in the latter the specified number of nodes along and across the basin would appear to require interchanging. Thus, for $n = 1$ and $m = 4$, with $\gamma \simeq 1/3$ for the basin in question, Eq. 34 shows very readily that the period $\tau_{4,1}$ would be approximately 1/4 of 110 secs or 27.5 secs and not 69 sec, as given by Joosting. On the other hand, for $n = 4$ and $m = 1$, the period $\tau_{1,4}$ would be 0.625 x 110 or 69 sec. The author seriously questions whether all the periods apparently observed by Joosting in his model were modes in accordance with the nodal combinations given in his Table 7. The same values in essence evolve more readily from the nodal combinations given in Table 9.

The modes given by the author in Table 1 and Fig. 2 are the harmonic sequence of possible pure transverse seiches for the case of $n = 0$ in Eq. 34, and were adopted because they are believed to be reasonably representative of the regime of oscillations most likely to exist at the end of the dock where the ship was berthed. One good reason for saying this is that the entrance to the dock happens to be located in the long side of the basin on the seaward side close to the basin end ($\simeq$900 ft) of our interest so that activating pulsations of wave energy entering the dock are directed transversely across the dock with maximum strength at this end of the long basin. The end of the dock nearest the entrance thus tends to behave partly as a quasi-basin in the transverse sense and its responses can be expected to be strongest to modes of oscillation in which the emphasis is on m rather than n in Eq. 34. Thus, assuming that an exciting frequency of from 54 sec to 57 sec were imposed on the dock, the writer's belief is that the response at the basin end in question would be of the $m = 2$, $n = 1$ category rather than the $m = 1$, $n = 5$ combination of Table 9 and would not differ greatly from an $m = 2$, $n = 0$ type oscillation across the end of the dock.

It will readily be apparent from Eq. 34 that with $\gamma = 1/3$ the influence of n in the factor $\left[m^2 + \gamma^2 n^2\right]^{-\frac{1}{2}}$ is but little over 10% of that of m, so that if n also is less than m, as in the right-hand columns of Table 9, the term $\gamma^2 n^2$ is comparatively small compared with m^2. No serious error is made therefore by taking $n = 0$ in these cases as was done in Table 1. As a specific example we note that the two-dimensional oscillation $m = 5$, $n = 4$ has a period $\tau_{5,4} =$ 21.3 sec, but if $n = 0$, $\tau_{5,0} = 21.9$ sec, which is not greatly different. The oscillation in this case is essentially the fifth harmonic of the fundamental transverse oscillation specified in the author's Fig. 2 and Table 1.

It will be seen from Table 9 that the periods of the two-dimensional oscillations given in the last three columns all accord reasonably well with the trinodal, quadrinodal and quinquinodal seiches portrayed in Fig. 2. The sextinodal and septuanodal oscillations could equally well be justified. Moreover the periods in the first four columns of figures of Table 9, although varied, all result from what is essentially a uninodal oscillation ($m = 1$) at the basin-end in question. Consequently, the general system of pure oscillations given in the author's paper is largely validated as applicable to our problem.

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[16] "The Mechanism of Seiches in Table Bay Harbor, Cape Town," by B. W. Wilson, Proceedings, 4th Conf., Coastal Engrg., Chicago, Ill., November, 1953; Council Wave Research, Berkeley, Calif., p. 52.

(b) Joosting maintains that his model tests have shown that the wave height in respect of the 110 sec oscillation is about 40% greater in the corner of the basin than at the location of the tide-gage, which is approximately 670 ft from the corner along the landward long side of the basin. This seems rather remarkable insofar as the 110 sec period applies to an $m = 1$, $n = 0$ type of two-dimensional seiche in which amplitudes should tend to be the same all along the long sides of the basin. Even if the oscillation accorded with a two-dimensional seiche of $m = 1$, $n = 1$ type, such as would give a period $\tau_{1,1}$ of 105 or 106 secs according to Joosting's Table 7 or Table 9, the theoretical indications are that the height in the corner would exceed that at the tide gage by only 6%. At any rate the adoption of the tide-gage values of amplitude for the fundamental sieche of 110 sec period is completely justified by the assumption of an $m = 1$, $n = 0$ type of oscillation.

TABLE 9.—PERIODS OF TWO-DIMENSIONAL SEICHES IN HARBOR BASIN

Number of Nodes	along (n)	1	2	3	4	5	1	3	2	3	4
	across (m)	1	1	1	1	1	2	2	3	4	5
Period $\tau_{m,n}$ (in secs.)		105	92	78	69	57	54	49	35	27	21

Joosting quotes South African and French model tests on the basin in question as showing that amplitudes of oscillations within the basin are directly proportional to the wave periods for constant input wave height at the wave generator at the location of the harbor approaches. He concludes that a ratio of 1 : 0.5 : 0.2 would have been preferable to the author's 1 : 0.75 : 0.55 for the relative amplitudes of the fundamental, second and fifth harmonic seiches. This argument would apply if in fact the heights of the relevant long-period waves entering the approaches to the harbor were actually equal. There seems no special reason, however, for making such an assumption of equality. The author's ratios are based on the concept of a wave spectrum in which spectral height tends to decrease with declining frequency. Because the period of the quinquinodal seiche (22 sec) is not far removed from the frequency band of visible storm waves, it is rational to expect that the height of long waves of this period would have been relatively greater than the height of long waves of 110 sec period. The same agrument would apply in respect of long-waves of 55 sec period. The concept of a wave spectrum in itself is entirely rational for it is known that the surge phenomenon always accompanies visible waves and swells reaching the afflicted harbor. For the particular occasion discussed (May 5, 1952) it has already been shown[17] that waves of 10 sec to 12 sec period with a possible height of 14 ft were probably prevalent in the far outer roadstead during the occurrence of surging. The author's amplitude ratios are admittedly conjectural, in the sense that the assumed wave spectrum itself is conjectural, but they are considered to be more realistic than the ratios advocated by Joosting.

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[17] "Research and Model Studies on Range Action in Table Bay Harbor, Cape Town," (reply to discussion), Transactions, South African Inst. of Civ. Engrg., Vol. 2, No. 5, May, 1960, p. 89.

(c) Joosting's query regarding the relative tightnesses of the mooring ropes cannot be answered with any degree of certainty. It is the author's recollection that among rope failures reported for the harbor in question the heaviest toll was in fore and back springs, suggestive of the likelihood that these ropes, in the positions shown in Fig. 1, are often overloaded in normal berthing operations on ships.

(d) The author will admit that Joosting's suggestion of a quantity $H' (= -\xi/\sigma)$ given in his Eq. 25 would give the presentation of the equations added meaning. The quantity ξ used by the author, however, is not without physical meaning because it can be considered to be the effective surge velocity of the water causing motion of the ship, operative over the length of the ship in its particular location.

(e) Joosting's belief that another factor, capable of taking account of the proximity of the ship to the quay and the sea-bed, should enter into Eq. 16 may possibly be correct. The factor $(\sinh k\,d - \sinh k\,s)/\cosh k\,d$, however, already considers some part of the influence of the water depth and the clearance s of the ship bottom above the sea bed. Also, the influence of the nearness of the quay would seem to be governable already to some extent in the equations by adoption of an appropriate value of virtual mass coefficient C_M, if we accept the assertion of Sommet[18] that its value of 1.1 to 1.2 for a ship surging in open water should be increased to from 1.2 to 1.4 for a ship flanking a quay.

(f) Joosting challenges the adequacy of the author's factor $(\sin k\,l)/k\,l$ in establishing the influence of the ship's length on the effective driving forces promoting movement of the ship, and claims that the formula given by his Eq. 26 is more correct insofar as it takes into account both the acceleration of the water and the buoyant forces on the ship of the water within the wave slope. The author and Mr. Joosting have crossed swords mildly on this issue elsewhere[19,20] but while the author admits that his version neglects the pitching motion of the vessel, he is unable to satisfy himself in regard to the complete accuracy of Joosting's formula owing to the lack of any published presentation of its derivation. Thus, Joosting's Eq. 26 does not appear to agree with his original version of it[6] and his later publication,[21] although stating it in the form of Eq. 1, provides no derivation.

The competing factors are plotted by Joosting in his Fig. 12 and are seen to be quite similar in trend and not too vastly dissimilar in value, which suggests that the pitching effect may not be too important. But before accepting Joosting's claim that he is necessarily right and the author wrong, let us examine the case sited ($k\,l = \pi$) for which ($\sin k\,l)/k\,l$ is zero and for which Joosting's factor C_1 is 0.245 according to his Fig. 12. As Joosting rightly observes, this condition implies equality between ship length 2 l and wave length and he argues therefore that the ship must necessarily respond to the wave slope in the manner of his Fig. 11. This, however, is wishful thinking because the position of the ship can be anything whatever depending on the period of the seiche in relation to the natural pitching period of the ship. For example, in the average water depth of 45 ft the period of seiche whose wave length would equal the length of the ship (350 ft) would be 9.2 sec. The free pitching period of the ship, however, could be expected to be of the same order (9), so that pitching resonance would result in the ship lagging the water movement by about 90° of phase angle. The ship would, thus, actually be horizontal for the situation shown in Joosting's Fig. 11. In consequence, the effective driving force on the ship would be nil because the center half of the ship would be subject to a positive wave slope, and the remaining end quarters to an equal and opposing negative slope. In these circumstances, the author's factor ($\sin k\,l)/k\,l$ would be right and Joosting's formula wrong. The likelihood of pitching resonance occurring at $k\,l = \pi$, in depths of water normal to most harbors, is difficult to escape for all but the largest sizes of commerical vessels, so that the author's allowance for the influence of ship length would at least seem to be a safe approximation to the truth.

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[18] Discussion of "The Troubled Waters of Table Bay Harbor," by J. Sommet, Transactions, South African Inst. of Civ. Engrg., Vol. 2, No. 7, July, 1960, p. 146.

[19] Discussion of "The Troubled Waters of Table Bay Harbor," by B. W. Wilson, Transactions, South African Inst. of Civ. Engrg., Vol. 2, No. 7, July, 1960, p. 143.

[20] "The Troubled Waters of Table Bay Harbor," (reply to discussion), by W. C. Q. Joosting, Transactions, South African Inst. of Civ. Engrg., Vol. 2, No. 7, July, 1960, p. 147.

[21] "The Troubled Waters of Table Bay Harbor," by W. C. Q. Joosting, Transactions, South African Inst. of Civ. Engrg., Vol. 1, No. 10, October, 1959, p. 211.

(h, i) The author would agree here with Joosting except insofar as the latter's Eq. 26 may still be a bone of contention.

(j) The author will concede that a solution to the differential Eq. 12, leads naturally to the form of Joosting's Eq. 27 rather than Eq. 21 and is preferable because Eq. 12 applies to a seiche of the form.

$$\eta = A \cos \frac{m \pi x}{B} \sin (\sigma t + \epsilon) \quad \ldots\ldots\ldots\ldots (35)$$

rather than Eq. 15, which, inadvertantly, was incorrectly stated.

The author's Eqs. 22 are not invalidated by this error, although a correction must be noted in Eq. 22(b) for the phase angle, as acknowledged under (l) below. The phase angles were not evaluated numerically for the problem of the paper and do not feature in any of its conclusions.

(k) Joosting's version of the author's Eq. 22(a) certainly provides a nice way of looking at it. In his Eq. 31 Joosting has further refined the equation into a very interesting and acceptable form.

(l) The author is pleased that Joosting has pointed up what was essentially a transcript error in the value of $\tan \alpha$ in Eq. 22(b). This should read in accordance with Joosting's Eq. 29. If the solution $X = X_1 \sin (\phi - \alpha)$ is used, however, the phase angle will be correctly given by

$$\tan \alpha = \frac{2\,\beta\left(\frac{\sigma}{\omega}\right)}{X^{n-1}\,\Delta(n) - \left(\frac{\sigma}{\omega}\right)^2} \quad \ldots\ldots\ldots\ldots\ldots (36)$$

rather than Joosting's Eq. 30. For n = 1 Eq. 36 reduces to the well-known phase relationship, $\tan \alpha = 2\,\beta(\sigma/\omega)/[1 - (\sigma/\omega)^2]$, for a linear damped oscillating system.

(m) On the basis of the foregoing replies to the various points made by Joosting, the author sees no reason to accept Joosting's contention that the effective amplitude of the fifth harmonic of the fundamental seiche was necessarily inadequate for exciting the critical ship motion. Joosting has put forward an interesting counter-presentation of the possible causes of trouble, but the author, while not disputing the possibility that coincidental seiches of longer period could also have been responsible for the breaking adrift of the ship, must reaffirm his conclusions as presented in his paper. The fact that other ships, moored in positions very close to that of the tanker in question, have responded to similar sea disturbances by surging at periods of from 20 sec to 30 sec has been established by actual measurement, in proof of which Joosting is referred to Table 2 in the author's paper,[22] which gave the results of measurements made during 1944 through 46.

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[22] "Research and Model Studies on Range Action in Table Bay Harbor, Cape Town," by B. W. Wilson, Transactions, South African Inst. of Civ. Engrg., Vol. 1, Nos. 6 and 7, June, July, 1959, p. 131.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 3224

SPACING OF INTERCHANGES ON FREEWAYS IN URBAN AREAS

By Jack E. Leisch,[1] M. ASCE

With Discussion By Gerald D. Love

SYNOPSIS

Location and spacing of interchanges on freeways has a pronounced effect on the transportation system of a city. Widely spaced interchanges do not provide the needed service; on the other hand, too many interchanges result in loss of speed and capacity. Bases for spacing of interchanges, including schemes for increasing operational efficiency, are presented.

INTRODUCTION

The ability of a freeway to carry traffic effectively depends to a great degree on the location and spacing of its interchanges. Widely spaced interchanges do not provide the needed service or develop the potential use of the facility. Too many interchanges, on the other hand, result in friction, inefficiency, and loss of speed and capacity. What, then, is the right spacing of interchanges on urban freeways?

To answer the question, certain key factors should be examined: external factors such as size of city, type of area, and street pattern; and internal factors, such as geometric features and operational characteristics of the highway. Knowing these factors, general minimum spacing can be prescribed, and

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arrangements permitting closer than general minimum spacing can be developed.

EXTERNAL FACTORS INFLUENCING LOCATION AND SPACING

The street pattern as well as the arrangement and frequency of arterial streets crossed by the freeway have a definite influence on the location and spacing of interchanges. Need of frequent ramp connections is minimized when the street pattern is regular, the familiar grid-iron, for example. This is especially true when the freeway parallels one set of streets and has continuous frontage roads. A heterogeneous or a disconnected pattern of streets together with an irregular arterial street system, on the other hand, tends to require close spacing of interchanges.

The location and frequency of freeway access connections vary somewhat with the size of the city as well as with topography and land use. Large commercial and industrial areas, because of the magnitude of traffic generation, call for numerous points of ingress and egress. This need obviously diminishes as the freeway passes outward from the central business district or away from areas of intensive development.

Interchanges are needed at local points of high traffic generation such as large industrial plants, airports, and stadiums or parks for sports events. Such interchanges, invariably of the directional type, need to be fitted into the normal sequence of interchanges. High traffic concentrations also occur where freeways merge or intersect. These points, too, call for elaborate interchange arrangements. The location of these major interchanges frequently affects the location of the freeways themselves as well as that of subsidiary interchanges. Large volumes of vehicles maneuver for position to leave or enter the freeway at these interchanges. Consideration must be given to proper signing and to the need for minimizing weaving of traffic. For these reasons, the distance from a major interchange to the next regular interchange should be greater than the spacing in a series of regular interchanges.

INTERNAL FACTORS INFLUENCING LOCATION AND SPACING

Operational characteristics of a freeway are governed largely by its geometric features. The more traffic that is imposed on a facility the more difficult and complex becomes its operation, particularly in conjunction with interchanges. Part of the problem can be overcome by expanding the facility laterally, that is, by adding width or lanes. But, much of the problem must be solved by expanding the facility longitudinally, in other words, by increasing lengths of maneuver areas and weaving sections. These play an important role in the spacing of interchanges.

The most troublesome feature of operation in conjunction with interchanges is that of weaving. This is produced when two traffic streams, moving in the same general direction, cross each other by merging and diverging maneuvers. Weaving is inherent to some types of interchanges, particularly the cloverleaf, but it is often produced by other types when they are closely spaced. Positive control of weaving through the spacing of ramp entrances and exits will be examined later.

Distances between interchanges are also affected by the necessary directional signing. There can be too many signs in a given length of highway. Drivers must have sufficiently long maneuver areas to read, comprehend, and act on the messages imparted. Thus, signing is a definite consideration in the spacing of interchanges.

SPACING OF INTERCHANGES ON EXISTING FREEWAYS

Having discussed factors influencing the location and spacing of interchanges, it is of interest to examine actual spacing on typical urban freeways. Table 1 shows such information for a few examples. Although this small sample may not be representative, it is at least indicative.

TABLE 1.—APPROXIMATE SPACING OF INTERCHANGES ON TYPICAL URBAN FREEWAYS[a]

Location (1)	Facility (2)	Approx. Length, in Miles (3)	Average Spacing Between Interchanges, in Miles (4)
Washington and Vicinity	Pentagon Network	4.3	0.35
	Shirley Highway	2.7	0.90[b]
Chicago and Vicinity	Congress Street Expressway	6.0	0.75
	Edens Expressway	13.0	1.30[b]
	Calumet & Tri-State Expressways	20.0	1.55[b]
Atlanta	Expressway (Route 295)	4.5	0.65
Detroit and Vicinity	Edsel Ford Expressway	5.3	0.55
	John C. Lodge Expressway	4.5	0.55
	Detroit Industrial Expressway	11.5	1.65[b]
New York and Vicinity	Parkways in Queens	18.0	1.00[b]
	Shore & Southern Parkways	22.5	1.70[b]
	New Jersey Route 17	7.3	0.90[b]
San Francisco	Bayshore Freeway	5.5	0.65
Philadelphia	Schuylkill Expressway	13.0	2.10[b]

[a] Distance between cross streets or intersecting highways to which ramps are connected.

[b] Suburban.

The approximate range and average of interchange spacing are shown in Table 2.

SPACING OF INTERCHANGES BASED ON OPERATIONAL FEATURES

The values given in Table 2 compare generally with the spacing of interchanges predicted on traffic requirements; that is, operationally there is a need for certain lengths of maneuver and weaving sections between successive interchanges. These lengths, of course, vary with the volume and pattern of through, entering and leaving traffic.

The absolute minimum spacing of interchanges may be based on a combination minimum maneuver and weaving length of 600 ft. See Fig. 1(a). An additional length of 600 ft on each side of this weaving section is estimated to be the minimum for an entering ramp on one end and an exiting ramp on the other end. The resulting spacing of 1800 ft is the absolute minimum that may be considered in urban areas involving light to intermediate weaving volumes. This spacing is equivalent to approximately 1/3 mile.

Normally the minimum weaving section between an entering and exiting ramp should be approximately 1200 ft. This is equivalent to two maneuver areas joined end to end, that is, a 600-ft merging maneuver length and a 600-ft diverging maneuver length butted together. Relatively heavy traffic can be accomodated, including weaving volumes upwards of 1500 vehicles per hour (vph). Assuming a length of 700 ft for each of the adjoining ramps, a minimum spacing of 2600 ft or about 1/2 mile is indicated, as shown in Fig. 1(b).

A preferable minimum spacing may be based on twice the normal length of maneuver areas, or a distance of 2400 ft between entering and exiting ramps, resulting in a span of about 4200 ft, or 3/4 to one mile. This is thought of usually as a favorable spacing in built-up areas.

TABLE 2.—LOCATION OF INTERCHANGES

Type of Area (1)	Range in miles (2)	Range in feet (3)	Average Spacing in miles (4)	Average Spacing in feet (5)
Urban	0.35-0.75	1,800-4,000	0.60	3200
Suburban	0.90-2.10	4,800-11,000	1.50	8000

These distances, of 1800 ft, 2600 ft and 4200 ft, are general guides of what may be considered the absolute, normal, and preferable minimums for spacing of interchanges on urban freeways. Another set of values, based specifically on the volume and pattern of interchanging traffic may be more appropriately used in design as criteria for spacing of interchanges. These values, as developed in Fig. 2, are predicted directly on the effects of weaving traffic ($W_1 + W_2$) entering traffic c-b maneuvering across leaving traffic a-d. Effect on operation of given volumes of weaving traffic varies with the length of weaving section e-f. Guide values are resolved as shown in Fig. 2 by considering weaving lengths over the entire range of weaving possibilities and coupling these with lengths in the range of 650 ft to 1100 ft for development of ramps on each end.

Spacings in the upper part of the tabulation, referred to as the low limit, are predicated on minimum weaving distances required to maintain normal operating speeds (35 mph) on a freeway.[2] Values in the lower part, designating high limits of spacing, are based on distances between entering and exiting ramps that are sufficient to place the traffic maneuvers out of the realm of weaving.[2] This information, including an intermediate set of values, is also depicted in Fig. 3 in the form of a design chart. Application of these criteria may be as follows:

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[2] "Policy on Arterial Highways in Urban Areas," Amer. Assn. of State Highway Officials, 1957.

Curve C, low minimum spacing, should be considered as a minimum control in highly built-up districts between ordinary interchanges, such as diamond and parclo (partial cloverleaf) type arrangements. This is not applicable in suburban areas except with C-D roads.

Curve B, high minimum spacing, may be taken as preferable minimum values in areas of concentrated urban development, and generally as the minimum values in suburban areas. Spacings shown by this curve may also be used appropriately as the minimum between a major interchange--directional type at the intersection with another freeway--and any ordinary interchange. The use of Curve B values as a minimum in this case is believed to be essential. More than normal distance is required to satisfy the complexity of maneuvers taking place in conjunction with major exits and entrances, and greater distances are required for advance overhead and other directional signing.

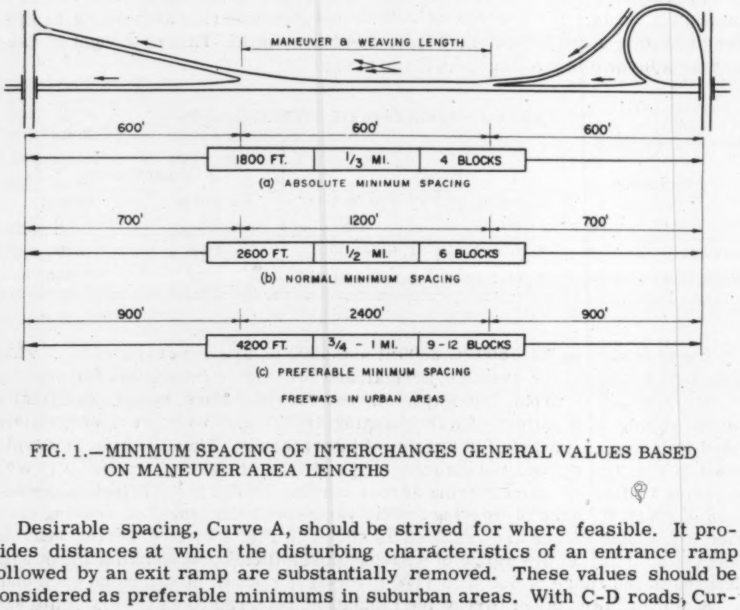

MANEUVER & WEAVING LENGTH
600'
600'
600'
1800 FT.
1/3 MI.
4 BLOCKS
(a) ABSOLUTE MINIMUM SPACING
700'
1200'
700'
2600 FT.
1/2 MI.
6 BLOCKS
(b) NORMAL MINIMUM SPACING
900'
2400'
900'
4200 FT.
3/4 - 1 MI.
9 - 12 BLOCKS
(c) PREFERABLE MINIMUM SPACING
FREEWAYS IN URBAN AREAS

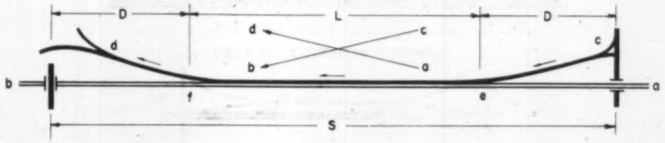
FIG. 1.—MINIMUM SPACING OF INTERCHANGES GENERAL VALUES BASED ON MANEUVER AREA LENGTHS

Desirable spacing, Curve A, should be strived for where feasible. It provides distances at which the disturbing characteristics of an entrance ramp followed by an exit ramp are substantially removed. These values should be considered as preferable minimums in suburban areas. With C-D roads, Curves B and A are desirable in suburban areas with intermediate local ramps.

ARRANGEMENTS PERMITTING CLOSER THAN MINIMUM SPACING

In suburban and outlying areas, spacing of interchanges to produce favorable operating characteristics can be accomplished without difficulty in most cases. In areas of concentrated urban development, however, proper spacing usually is difficult to attain because of traffic demand for frequent access. This comes about as a result of large volumes of traffic generated within a relatively small area, as in the vicinity of the central business district. There is a limit to the amount of traffic that can be accomodated per ramp; consequently, numerous ramps often are called for on freeways serving downtown areas. Sometimes the traffic demand for ingress and egress is of such magnitude that, if fully accomodated, the free-flowing characteristics of a freeway would be destroyed by too close spacing of ramps.

This presents a problem on the part of the designer to satisfy the two classes of traffic--through traffic and that terminating or originating in the business district. Each hinders the other under heavy volume conditions. To satisfy both, it becomes necessary to use special arrangements of ramps permitting more exits and entrances than can conventionally be provided. Such arrangements are shown in Fig. 4.



LIMITS OF SPACING	$(W_1 + W_2)$ weaving volume vph	D length allowed for each ramp feet	L* length of weaving section feet	S interchange spacing feet
LOW LIMIT (minimum)	1000	650	500	1800
	1500	850	900	2600
	2000	1000	1400	3400
	2500	1100	2000	4200
HIGH LIMIT (desirable)	1000	850	2300	4200
	1500	1000	4000	6000
	2000	1000	6000	8000
	2500	1000	8000	10,000

* From AASHO Policy on Arterial Highways in Urban Areas, 1957:
Minimum based on relations for 35 mph average running speed, p. 492;
Desirable based on distances at which traffic maneuvers pass out of realm of weaving, p. 493.

FIG. 2.—LIMITS OF INTERCHANGE SPACING BASED ON WEAVING MANEUVERS

Ramps serving a large traffic generator, such as a downtown district, need not be thought of as parts of individual or distinct interchanges. Instead, they may be considered as a series of exit and entrance ramps serving the area. For example, ramp frequency can be increased by criss-cross or nested arrangements as shown in Fig. 4(b). Normal sequence of exits and entrances of Fig. 4(a) is maintained here, but a greater number of ramps is made possible by the overlap.

Increase in number of ramps also may be effected by grouping as indicated in Fig. 4(c). Two or more successive exits followed by two or more successive entrances obviously permit the use of a greater number of ramps in a given length of highway than by the usual method of alternating entrance and exit ramps shown in Fig. 4(a).

Another method by which the frequency of connecting ramps may be increased is by the use of collector-distributor roads, illustrated in Fig. 4(d).

Because ramps connect to the C-D road rather than to the through traveled way, their number may be increased due to shorter maneuver and weaving area lengths required. But, what is more important is that through and terminating vehicles are separated, permitting freeflowing characteristics to be maintained on the freeway proper.

Large volumes of traffic also may be discharged and collected along a relatively short length of freeway by the use of a lateral distributor as shown in Fig. 4(e). The distributor is a lateral extension of the freeway, in effect, coupled with a series of ramps connecting with the street system. By this arrangement, one major exit and entrance along the freeway can accomodate numerous ramps.

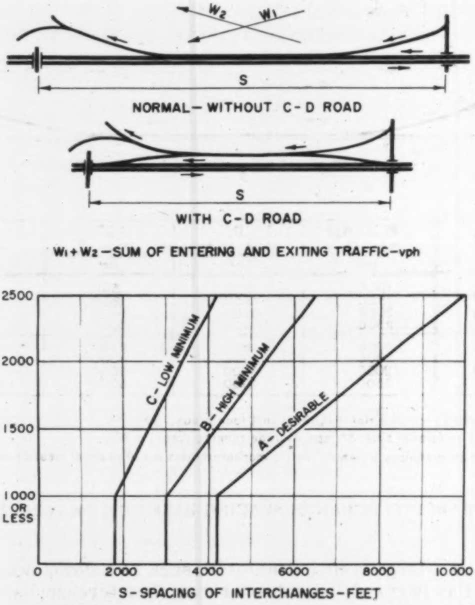


FIG. 3.—DESIGN CHART FOR SPACING OF INTERCHANGES AS CONTROLLED BY OPERATIONAL FEATURES

The extent to which ramp frequency may be increased by these devices or combinations thereof is illustrated in Fig. 5. The numbers at the right show the ramp frequency index, which is the ratio of the number of ramps per mile on a given arrangement to the number of ramps per mile on the conventional or basic arrangement (a). The arrangement (b) with criss-cross or nested ramps has an index of 1.2, or an increase of 20% in the number of ramps over arrangement (a). Plan (c) with ramp groupings and plan (d) with a C-D road both have a ramp frequency index of 1.3. Arrangement (e), combining nested and groupings of ramps, increases the number of ramps by 50%. Arrangement (f), combining a C-D road and ramp groupings, increases the number of ramps by 70%. Using lateral distributors in series, plan (g), the number of ramps may be twice that of the conventional plan (a).

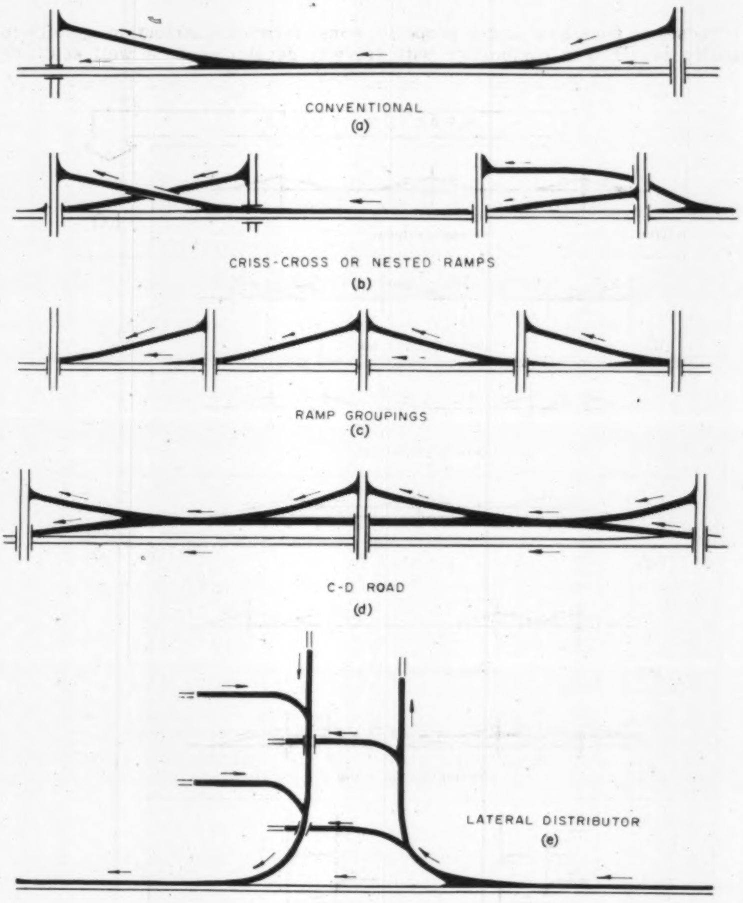

CONVENTIONAL
(a)
CRISS-CROSS OR NESTED RAMPS
(b)
RAMP GROUPINGS
(c)
C-D ROAD
(d)
LATERAL DISTRIBUTOR
(e)

FIG. 4.—ARRANGEMENTS TO INCREASE FREQUENCY OF RAMPS

These various arrangements are useful devices for increasing the overall ability of a freeway to discharge and collect traffic in a relatively short distance without decreasing the capacity or the effectiveness of the freeway.

DISTRIBUTION SYSTEMS

To serve downtown areas properly, some form of distribution system is usually essential in conjunction with freeway development. A well designed

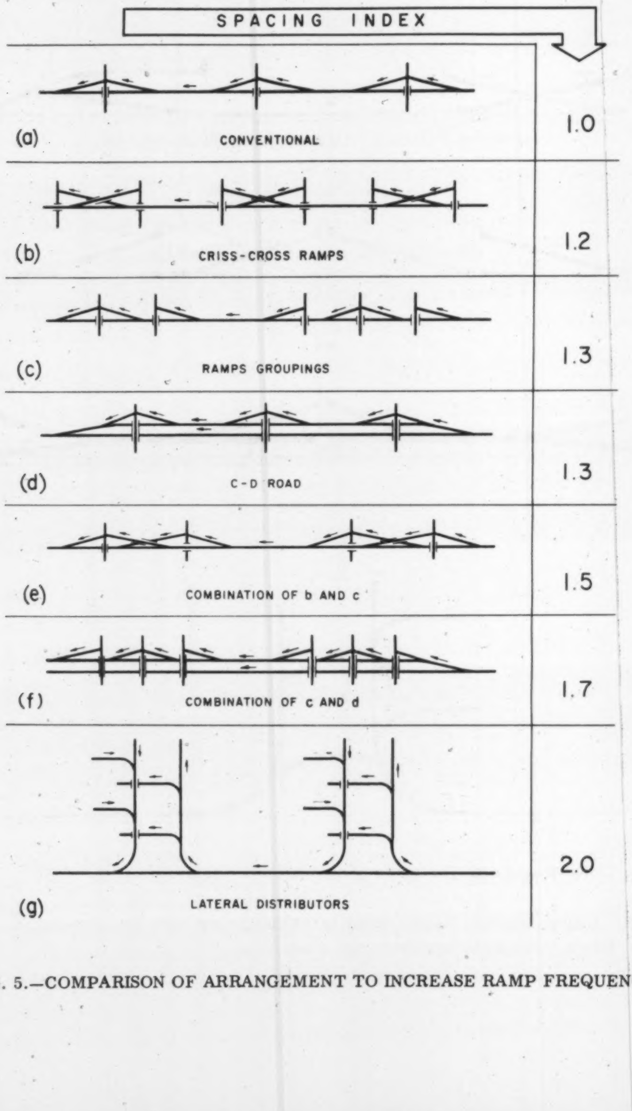
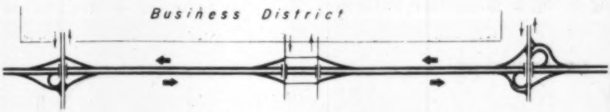

SPACING INDEX
(a) CONVENTIONAL
1.0
(b) CRISS-CROSS RAMPS
1.2
(c) RAMPS GROUPINGS
1.3
(d) C-D ROAD
1.3
(e) COMBINATION OF b AND c
1.5
(f) COMBINATION OF c AND d
1.7
(g) LATERAL DISTRIBUTORS
2.0

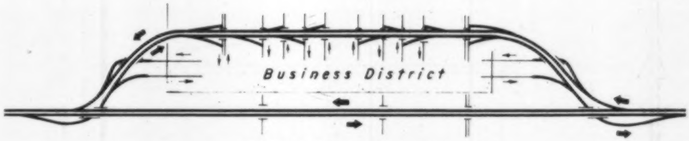
FIG. 5.—COMPARISON OF ARRANGEMENT TO INCREASE RAMP FREQUENCY

system assures efficient operation, not only on the freeway itself but on the surface streets serving it. The devices for increasing the ability of a freeway to discharge and collect traffic are also applicable to distribution systems. Furthermore, a network of feeder streets, preferably providing one-way operation, is a necessary adjunct. These two features are combined to make up various forms of distribution systems as illustrated in Figs. 6, 7, and 8.



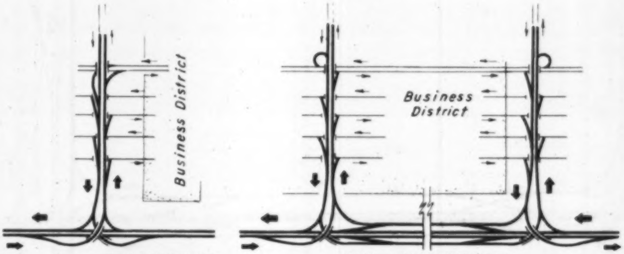
INDIVIDUAL INTERCHANGES

(a)



PARALLEL DISTRIBUTOR

(b)



TRANSVERSE DISTRIBUTOR
(c)

DUAL TRANSVERSE DISTRIBUTOR
(d)

FIG. 6.–DISTRIBUTION SYSTEMS–DIAGRAMMATIC

A single freeway skirting a downtown area may be served by arterial cross streets and individual interchanges, as shown in Fig. 6(a). Sometimes interchanges must be closely spaced, however, and the traffic terminating or originating in the business district is so great as to overload the ramps and connecting streets. In such cases, the problem may be solved by one of the other arrangements in Fig. 6 or Fig. 7. The appropriate design depends on the size and shape of the downtown area and the location of freeways.

Fig. 6(b) diagrams one arrangement in conjunction with a single freeway that can handle a tremendous volume of traffic to and from the business district. The parallel distributor is a controlled access facility, much like a service siding along a main-line railroad. A highly efficient system is developed for both through and local traffic by means of two directional Y-interchanges, a series of ramps, and a network of one-way streets.

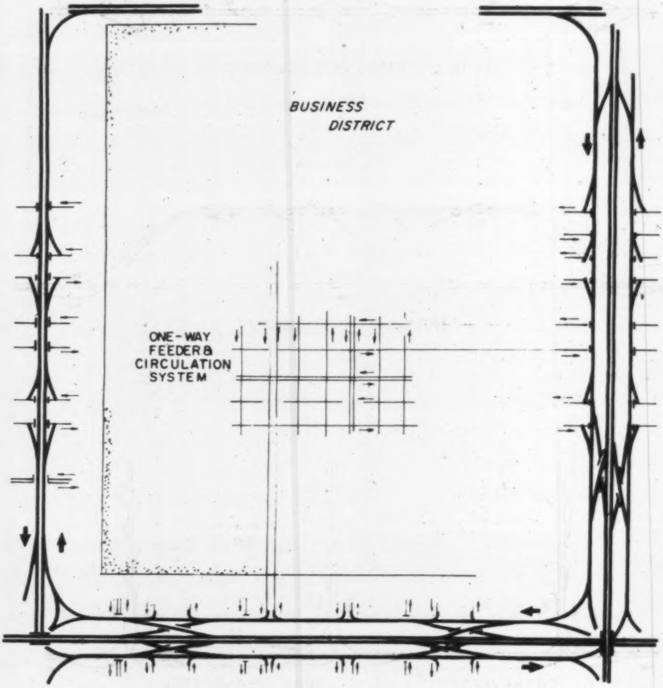
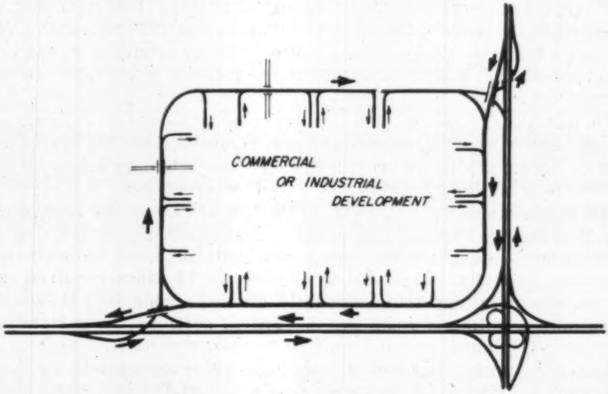


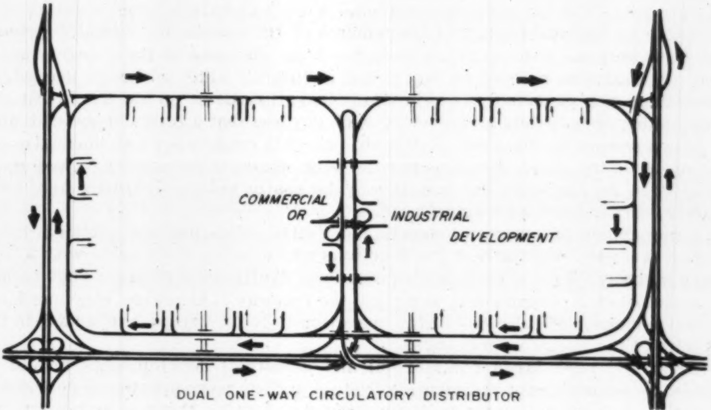
FIG. 7.—INNER CIRCUMFERENTIAL HIGHWAY

The transverse distributor shown in Fig. 6(c) is suitable for a downtown area of nominal size with a freeway located at right angles to its long dimension. The distributor is a freeway spur of any length required to handle the discharge and pick-up of traffic. Ramps connect with a series of one-way streets into and out of the business district.

For larger downtown areas, two such transverse distributors may be utilized as indicated in Fig. 6(d). This provides a highly flexible and efficient arrangement. The arterial street at the top of the diagram completes a circumferential with only one leg without control of access. C-D roads may be used along the freeway between the major interchanges, as shown, to alleviate weaving and to permit through traffic to operate freely.



ONE-WAY CIRCULATORY DISTRIBUTOR
(a)



DUAL ONE-WAY CIRCULATORY DISTRIBUTOR
(b)

FIG. 8.—DIAGRAMMATIC DISTRIBUTION SYSTEMS FOR OUTLYING COMMERCIAL AREAS

In large metropolitan areas, the arterial system of highways generally calls for a belt freeway around the central business district. The two-way inner loop permits any point within to be reached with minimum travel distance. Necessary adjuncts to this elaborate distribution system include a one-way feeder and circulation network of streets, and arrangements of ramps to permit high discharge and pick-up capacity.

Radial freeways in large cities, often overlap the inner belt highway, resulting in large accummulation and weaving of traffic. Future traffic is frequently estimated at daily volumes of 100 thousand to 150 thousand vehicles on parts of such a facility. These volumes often call for unwieldy widths of pavements and unworkable weaving sections if the freeway is designed as one pair of roadways.

In such instances, however, the problem can be solved on a single facility if the troublesome weaving maneuvers are removed from the through lanes. This may be accomplished by providing C-D roads continuously on all or parts of the inner loop highway between the major interchanges. As shown on the bottom and right legs of the loop (Fig. 7) the through lanes are free of the disturbance of entering, leaving, and weaving traffic. Thus, by the use of C-D roads, the inner lanes of the freeway retain their express characteristics of high speed and capacity. The total of 12 lanes or 14 lanes required on parts of the circumferential can thus be readily divided among four traveled ways.

Without such doubling of roadways on critical portions, the system is apt to be thrown out of balance, causing serious breakdowns during peak hours. In large cities, for example, the radial highways on the approach to the inner belt are invariably 6 lanes and 8 lanes wide. For short lengths, where auxiliary lanes are used for exits and entrances, they become 8 lanes and 10 lanes wide. This normally is considered the limit for the roadways constituting the freeway proper. At least one additional lane in each direction is often called for, however, by the overlapping of movements of the converging radial freeways and the additional ramps required on the belt. Because of the rule-of-thumb limit on maximum number of lanes, this additional width, although needed, is sometimes not provided. The omission is rationalized on the basis that the overloading occurs only a few hours each day and that a fully adequate design is not economically justified. Failure to use C-D roads where an unusual number of lanes is called for, however, is poor design and may be looked on as false economy, because the result may be costly delays to traffic and, ultimately, construction of a relief road.

Commerical or industrial development often takes place in outlying areas along a freeway, especially at intersections with another freeway or with a primary highway. Such areas may become huge traffic generators. They should be anticipated in planning and designing the freeway. Otherwise, they can later cause serious difficulty in traffic operation if local ramps are added in the vicinity of the major interchanges.

Properly, such large commercial and industrial developments should be served by some form of distribution system such as those previously described. In outlying areas, unlike the built-up districts of cities, there is an opportunity to develop one-way circulatory distributors, as illustrated in Fig. 8. The one-way clockwise distributor, because of lack of an existing street system, can be readily made a controlled access facility. The highway can be built at existing grade with only an occasional grade separation where interconnection with adjoining areas is required.

In Fig. 8(a), the two interchanges serving the area are properly spaced from the major interchange. The interchanges with the distribution system are simple and direct as a result of the one-way clockwise movement on the distributor. Although the system provides high capacity and considerable flexibility of movement, it has little adverse effect on through traffic. Some movements into and out of the area necessitate longer travel distance than would be required on a two-way circulatory distributor. However, if the latter were provided as a controlled access facility, it would likely be prohibitively costly. A two-way circulatory distributor without control of access also would be less desirable than the plans of Figs. 8(a) and 8(b) because of at-grade conflicts and generally less efficient operation.

The builders of such a commercial or industrial development should ordinarily provide the traffic distribution system as part of the project. Generally not more than one such large area should be permitted to develop at the intersection of two freeways. If positioned in diagonally opposite quadrants, however, two such areas may be adequately served. The general plans of Figs. 8(a) and 8(b) are also applicable to development of future neighborhood units in outlying areas. These plans can be readily adapted to stage construction, spreading the cost over a long period of time.

CONCLUSION

Location and spacing of interchanges on freeways has a pronounced effect on the transportation system of a city. Interchanges widely spaced do not provide the needed service or develop the potential use of the arterial system of highways. Too many interchanges on the other hand, lead to inefficient operation and tend to destroy the free-flowing characteristics of controlled access facilities.

It is desirable to space freeway interchanges widely when considering land use and operational factors. In order to fit into the pattern of existing or proposed developments, however, interchanges frequently must be spaced closer than otherwise deemed appropriate. This is especially true in providing connections between freeways and the distribution networks serving central business districts or other heavy traffic generators.

This paper offers certain guides to freeway designers, but in so doing, it barely introduces a very important subject. There is an urgent need for highway, traffic, and planning engineers to combine their efforts in conducting appropriate research on this vital phase of planning and design of urban freeways.

DISCUSSION

GERALD D. LOVE,[3] **A. M. ASCE.**—The spacing of interchanges on freeways is one of the most important aspects of urban freeway design. As Leisch so ably points out, traffic service tends to be reduced when the spacing becomes too great and when too many interchanges are provided, the operational efficiency of the freeway itself is compromised. These two features, traffic service and operational characteristics, establish the limits of maximum and minimum interchange spacings.

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The need for weaving sections of certain lengths between successive interchanges to satisfy traffic operational requirements, represents a valid approach to the determination of minimum interchange spacings. Leisch's "Design Chart For Spacing of Interchanges" has been developed on this basis. The two related variables of traffic flow, speed and volume, determine the required length of weaving sections. Thus, a weaving section of a certain length can accommodate different volumes of traffic if the speed is varied. This basic speed-volume information is essential in relating the operation of weaving sections to the overall operational characteristics of the freeway. Although Leisch makes reference to the fact that curve "C," which is labeled "low minimum spacing," is based on an average running speed of 35 mph, there is no indication as to the operating speed associated with curve "B." The design chart, presented as Fig. 3, would be more useful if the curves were developed for various operating speeds in lieu of curves labeled with such descriptive phraseology as low minimum, high minimum, and desirable. Any series of curves developed for interchange spacing would be essentially the same type as the weaving section curves presented in the Highway Capacity Manual.

Normally, weaving sections for collector-distributor roads are designed on the basis of a somewhat lower operating speed than weaving sections forming an integral part of the freeway. This aspect is brought out by Leisch in his proposal to utilize C-D roads as a method by which the frequency of connecting ramps may be increased. The use of the design chart would, thus, tend to provide a very liberal design. It does not appear that curve "C" represents a realistic minimum for determining interchange spacings involving collector-distributor roads.

Some interesting arrangements for permitting closer than so-called minimum spacing of ramps have been presented. As the author points out, the traffic demand for ingress and egress in some instances is of such magnitude that, if fully accommodated, the free flowing characteristics of a freeway would be destroyed by spacing ramps too closely. There are certain advantages to be gained by the use of C-D roads and criss-cross ramps. Their use, however, as a means of permitting closer spacing of interchanges, per se, is a moot question. Although certain operational advantages can be gained by various ramp arrangements, ramp capacities are dependent, to an important degree, on the volume of traffic on the freeway. Increasing the ramp frequency may result in an unbalanced design. By providing a large number of points of ingress and egress, there is a greater possibility that the freeway will break down as a result of the previously mentioned critical traffic densities. In some instances this difficulty might be overcome by increasing the capacity of the freeway by added lanes but, as pointed out by Leisch, there is a practical limit to the number of lanes that can be provided so this solution can seldom be applied. It would seem that the primary consideration in determining the number of points of ingress and egress in areas of concentrated urban development would be that of a balanced design whereby a sufficient number of ramps are provided to permit the freeway to operate at practical capacity. With this basic design approach, ramp capacities are brought into balance with the capacity of the freeway, thus, preventing breakdowns of urban expressways because of overload. Traffic patterns, of course, must be considered to establish the inter-relationship of through freeway traffic and traffic volumes entering and exiting from the various ramps.

Although lateral distributors may be very effective in collecting and distributing traffic, the utilization of this type of design as an arrangement to increase the frequency of ramps is questioned. The spacing that is being considered is the interchange or ramp spacing along the freeway. The same design criteria (minimum weaving distance requirements) control the spacing of lateral distributors shown in Fig. 5(g) as in the conventional layout shown in Fig. 5(a). Apparently, the author derived the spacing index of 2.0 for the lateral distributors by some other means.

Leisch's concept of providing C-D roads continuously on all or parts of inner loop highways between major interchanges provides a means of removing weaving maneuvers from the through lanes. A diagrammatic sketch of an inner circumferential highway was presented as Fig. 7. Some of the major disadvantages of this proposal may be visualized by referring to this sketch. Multi-level interchanges are shown at the two intersections of the freeway and the distributors. Interchanges of this type require considerable right-of-way, which is very costly in downtown areas. The problems associated with obtaining satisfactory grades and the elimination of the existing surface streets in the interchange areas are of major concern. To maintain the continuity of the existing street network it would be necessary to either elevate or depress the freeway and C-D roads. Although weaving movements can be removed from the freeway by providing continuous C-D roads, the heavy weaving volumes on the C-D roads would still be a major problem. The construction of a total of 12 or 14 lanes on four traveled ways with provisions for ramp connections between parallel roadways would require an extremely wide right-of-way. These features have been presented to support the point that freeways with more than 6 or 8 lanes in areas of concentrated urban development involve designs of doubtful practicability from both an engineering and economic viewpoint. The resulting heavy concentration of traffic may also lead to serious difficulties.

What other means or types of designs, then, may provide a solution to the movement of heavy traffic flows in downtown areas? If at all possible we should avoid an extreme concentration of traffic on one facility with the resulting interchange and distribution problems. Approximately 30% of downtown areas are occupied by streets and parking areas. Even at the time of peak hour flows the potential street capacity is not fully utilized in our large metropolitan cities. Although traffic destinations may be concentrated in the CBD, there are many streets in this area to absorb the demand.

A logical approach to the problem would be to increase the capacity of the existing facilities by means of traffic control devices, one-way streets, the elimination of parking, at least during peak hour flows, and the construction of additional at-grade expressways. Freeways and inner circumferentials should, of course, be considered as an integral part of an over-all city highway improvement program but the number of lanes could be limited to six or eight if additional capacity could be obtained by improving the existing surface street network.

It would seem that in some instances, multi-lane freeways have been proposed as a solution to the growing traffic problems in our large urban areas. Certainly, freeways have a definite function but the over-all traffic problem cannot be solved by freeways alone. Better utilization of our existing street networks must be considered in meeting our growing traffic demands.

Leisch has presented a very interesting paper on a subject of utmost importance. The problem of properly serving the traffic demands of downtown areas should receive the cooperative attention of city and state representatives.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3228

USE OF MATHEMATICAL MODELS IN ESTIMATING TRAVEL

By Alan M. Voorhees,[1] M. ASCE

SYNOPSIS

On the basis of recent research on urban travel habits, it is now feasible to estimate urban travel patterns by mathematical procedures. These techniques can be used to predict highway volumes as well as number of passengers that will use a specific mass transit improvement. Tests of these procedures indicate a high degree of reliability.

INTRODUCTION

The development of any new techniques usually brings along with it some new jargon. In the field of traffic planning, the new word is "models." Had these techniques been developed in the 1940's or 1930's, they would have simply been called "formulas," but to conform to other professional groups, these procedures are referred to as mathematical models.

A model, in a descriptive sense, utilizes certain mathematical techniques that involve various steps and formulas. More simply, it can be defined as a mathematical statement of observed relationships. For example, surveys of shopping habits have revealed that shoppers follow certain patterns that are related to the size of the shopping center and inversely related to the travel time to them. When such mathematical relationships are established it is possible to estimate or forecast where people will shop. When these techniques deal with travel habits, they are known as traffic models.

Now it becomes apparent from the analysis of origin-destination (O-D) studies that have been done by the United States Bureau of Public Roads, Dept. of Commerce (BPR) and others, that traffic movement in urban areas can be

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simulated by an application of a traffic model.[2] In other words, if you know certain factors about a community, such as the number of people that live and work in various zones, number of trips per car, trip length characteristics as well as the car ownership in these zones, it is possible, by following certain mathematical procedures, to estimate the O-D of urban travel.

To understand a traffic model, an examination of how a model could be applied might be helpful.

The method of synthesizing traffic is generally carried out in two steps. First, the frequency and mode of travel of individual trips are determined and then, the destination of these trips is estimated.

The frequency of these trips naturally depends upon individual needs and desires. For instance, an insurance salesman undoubtedly travels more each day than a typical industrial employee. The mode of travel either party chooses will depend on the availability of an automobile, the adequacy of public transportation services, or both.

The destination of these trips will depend on the distribution of land uses—the location of shopping centers, industrial and commercial areas, and residential neighborhoods. In a way, it might be said that individual travel patterns reflect the opportunities that are afforded.

TRIP FREQUENCY

In advance of estimating trip frequency, a decision must be made on the number of trip purposes that should be used. At the present time it is generally agreed that from two to four categories of trips should be used depending on city size. In the Baltimore Transportation Study four trip purposes were used.

1. Work trips (to and from work): Does not include repair runs and salesmen trips.
2. Commercial trips: To and from commercial areas including trip for personal business, shopping, and dining or entertaining.
3. Social trips: To residential areas including trips to school and visiting friends.
4. Non-home based trips: Consist, for example, of salesmen in door-to-door traveling or housewives shopping from store to store.

In computing trip frequency, it is very helpful to have a small number of home interviews to establish these patterns. Usually commercial- and social-trip frequency were based on car ownership. For example, in Baltimore, Md., it was found that for every 1,000 cars garaged in a residential area, 800 commercial and 400 social auto trips commenced daily and of these 40 commercial and 15 social trips started at the peak hour.

Computing work-trip frequency usually involved a more complicated procedure because the number of work trips is related to employment rather than car ownership. In Hartford, Connecticut, this was done first by estimating the number of workers usually departing daily from a residential district by some form of transportation (85% of labor force); then the number of transit work trips starting from a residential areas was based on Table 1. The total number of transit trips should then be computed and compared with the number of transit trips made in the area each day. (This should be about one quarter of the revenue passengers because about one half of all transit trips are related to work.) If the comparison is not favorable, a comparison should be made on a sectional basis of the number of buses in and out of an area with the estimated number of transit work trips. Then appropriate adjustments should be applied.

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[2] "Travel Patterns in 50 Cities," by Frank B. Curran and Joseph T. Stegmaier, Bulletin 203, Highway Research Bd., Washington, D. C., 1958.

TABLE 1.—PERCENTAGE OF ALL WORK TRIPS MADE BY TRANSIT[a]

Cars per 1000 persons	Net Land per Family - in square feet					
	10,000	5,000	2,500	1,200	600	300
500	5%	7%	11%	19%	33%	65%
450	7	9	13	21	35	67
400	9	11	15	23	37	69
350	11	13	17	25	39	71
300	13	15	19	27	41	73
275	14	16	20	28	42	74
250	15	17	21	29	43	75
225	16	18	22	30	44	76
200	17	19	23	31	45	77
175	18	20	24	32	46	78
150	19	21	25	33	47	79
125	20	22	26	34	48	80

[a] Based on Highway Traffic Estimation, Eno Foundation, and Data from the Chicago Area Transportation Study.

TABLE 2.—RELATIONSHIP BETWEEN PERSONS PER CAR AND CAR OWNERSHIP FOR WORK TRIPS

Cars per 1,000 persons	Persons Per Car
100	1.65
150	1.52
200	1.46
250	1.40
300	1.33
350	1.30
400	1.27
450	1.23
500	1.20

Based on: Washington, D. C. Transportation Survey.

The number of work trips made by auto can be determined by subtracting the transit work trips from the number of workers departing by some form of transportation. To figure the number of automobile-driver work trips starting from each area, the number going by auto should be corrected for car pooling, see Table 2.

The frequency of non-home based trips can be established in several ways. In Hartford, Connecticut it was determined that 1,200 trips start each day for every 1,000 cars and 30 commence at the peak hour. Furthermore, the number of trips starting from each zone is in direct proportion to the number of people employed in each zone plus five times the retail employees. This is in line with the fact that about half of such trips start from commercial areas and the rest are related to place of employment.

To keep the technique simple, only two types of transit trips need to be considered, work trips and miscellaneous trips. The frequency of miscellaneous transit trips usually is equal that of work trips, and thus it is possible to estimate all transit trips from Table 1.

TRIP DESTINATION

The destinations of these various types of trips can be determined by several methods. In Baltimore, Md., the so-called gravity model was applied. Essentially, this model follows Newton's law of gravity and states that all trips emanating from an area are attracted or "pulled" to various land uses. The strength of this pull is directly related to the size of land use development and inversely related to the distance (or travel time) between the land use and the starting point of the trip.[3]

Under this concept for transit travel, transit time between zones is used, whereas auto-travel time between zones is applied for private-vehicle trips.

This reflects the fact that transit users tend to adjust their traveling habits in accord with mass public transportation service. Travel time between zones depends on transit service. The auto user, being more versatile, is influenced by the travel time permitted by the highway network. Because most auto and transit travel occurs in the off-peak hours, midday hours were selected for study.

To appropriately select suitable factors to express the "size" of the attracting land use for work trips, the total number of people employed in each area is usually analyzed. For commercial trips, retail employment for each zone may be examined to reflect the size of the attractor. This index is often selected because it may be easy to obtain retail employment information. In considering social trips, the number of people living in each area is generally chosen to indicate attractor size.

For non-home based trips, employment plus five times retail employment may be used. The exact factor or factors that should be used well depend on the availability of statistical data and the appropriateness of such factors.

Using employment statistics and population figures made it easier to link the survey with the economic-base data for the area. From experience gained in New Haven, Connecticut, it appears that the size of the attractor is more effectively expressed in terms of employment and population rather than in acreage of various land uses.

The effects of travel time on trip destination may be measured by a series of factors like those shown in Table 3. These travel-time factors can be determined from a small number of interviews. Since these factors vary by trip purposes, they should be established for various trip types.[4,3] In effect, these factors indicate the influence that travel time has on travel patterns.

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[3] "A General Theory of Traffic Movement," by Alan M. Voorhees, Inst. of Traffic Engrs., 1955.

[4] "Predicting Local Travel in Urban Areas," by J. Douglas Carroll, Jr. and H. W. Bevis, Papers and Proceedings, Regional Science Assn., 1955.

As an example of the significance of these factors it can be shown that an industrial zone two minutes from a residential area attracts four times as many work trips as a comparable industrial zone 10 min distant, see Table 3.

To compute the destination of work trips beginning in a residential zone, the appropriate time factor is multiplied by the number of people employed in various zones. Work trips are subsequently distributed to each employment zone in proportion to that zone's product and the sum of the product for all zones. (An example of this process will subsequently be presented.)

TABLE 3.—TRAVEL-TIME FACTORS USED IN THE BALTIMORE TRANSPORTATION STUDY

Travel Time In Minutes (1)	Relative Frequency of Trips by Type			
	Work (2)	Social (3)	Commercial (4)	Non-Home Based (5)
2	4.00	5.00	8.0	8.0
3	2.86	3.33	7.0	7.0
4	2.28	2.50	6.0	6.0
5	1.90	2.00	4.0	4.0
6	1.60	1.62	2.7	2.7
7	1.40	1.42	2.0	2.0
8	1.21	1.25	1.5	1.5
9	1.11	1.11	1.2	1.2
10	1.00	1.00	1.0	1.0
11	0.93	.91	.80	.80
12	.86	.83	.68	.68
13	.80	.77	.57	.57
14	.75	.71	.50	.44
15	.70	.67	.44	.40
16	.66	.62	.40	.35
17	.62	.59	.35	.32
18	.59	.55	.32	.28
19	.56	.52	.28	.25
20	.53	.50	.25	.22
21	.50	.46	.23	.19
22	.47	.43	.21	.16
23	.44	.40	.20	.13
24	.41	.37	.18	.10
25	.39	.34	.16	.08
26	.36	.32	.15	.06
27	.33	.30	.14	.04
28	.31	.28	.13	.02
29	.27	.26	.12	.01
30	.25	.25	.11	-
32	.21	.21	.10	-
34	.18	.18	.08	-
36	.16	.16	.07	-
38	.14	.14	.07	-
40	.12	.12	.07	-
46	.07	.07	.05	-
50	.04	.04	.03	-
60	.01	.01	.01	-

This same general technique may be used in studying truck travel in an urban area, for various studies have shown that the non-home-based trip pattern is fairly comparable to truck movement patterns in urban areas.

When the gravity model is used to distribute work trips, the distribution must be brought into balance. For example, if, in its application, too many trips were allocated to a particular employment center, they should be adjusted to conform to the estimated number of auto and transit trips destined to a center.

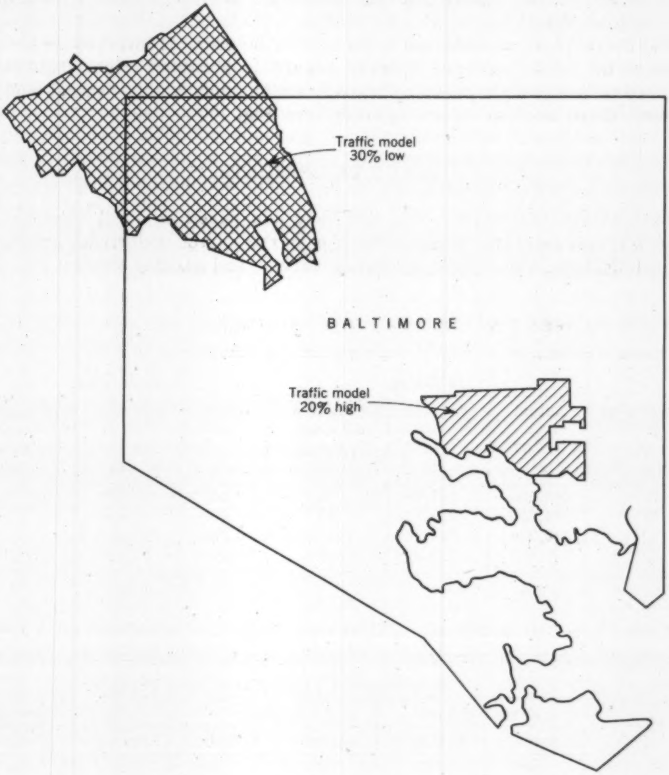

FIG. 1.—RESIDENTIAL AREAS WHERE OBSERVED TRAVEL TO DOWNTOWN BALTIMORE VARIES FROM TRAFFIC MODEL

This can be achieved by multiplying the trips to the center by an appropriate adjustment factor. These corrective measures should be made to work trips only.

To compute the number of auto or transit work trips destined to an area, Table 1 can be used. This reflects the fact that transit usage, in an area of low car-ownership, naturally will be high. Conversely, in an area of high car-ownership transit travel will be low. Hence, Table 1 can be used to indicate this relationship, and applied in developing the necessary computations.

This model can also be modified for trips to the downtown area. This may be necessary to adjust for the difference in relationships between home and place of employment for different occupational classes. (From experience in other cities it would appear that this correction is only necessary for trips to the downtown area.)[5]

The correction for downtown trips is not complex. It involved an investigation of the model's degree of error regarding downtown trips. This can be achieved by analyzing a previous transportation study for the central business district (CBD) such as a parking study (see Fig. 1).

CHECK THE RESULTS

Even though the factors that are used are based on local observations, a check should be made. Some of the checks that can be undertaken are screen line comparison, employee's residence survey, and parking studies.

TABLE 4.—SCREEN LINE CHECKS OF THE BALTIMORE 1958 TRAFFIC ESTIMATES

	24 Hours			Peak Hour		
Screen Line (1)	Actual (2)	Estimate (3)	Estimate As Per Cent of Actual (4)	Actual (5)	Estimate (6)	Estimate As Per Cent of Actual (7)
A	487,500	457,200	94	40,100	40,300	100
B	384,900	399,100	102	35,400	32,900	93
C	323,200	365,700	112	30,200	33,700	111
D	254,400	280,900	110	23,000	24,100	105

TABLE 5.—COMPARISON OF ACTUAL AND ESTIMATED NUMBERS OF TRIPS[a]

Time of Trip in Minutes	Percentage of Trips Within Time Periods					
	Westinghouse		Bendix		Glen L. Martin	
	Actual	Estimate	Actual	Estimate	Actual	Estimate
0 - 10	42	37	32	29	-	-
0 - 20	64	60	74	76	24	17
0 - 30	82	78	94	94	71	64
0 - 40	97	98	100	100	93	94
Over 0. . . .	100	100	100	100	100	100

[a] By means of Travel Time from three Manufacturing Plants, Baltimore, 1958.

In Baltimore, four screen lines were created, that divided the metropolitan area into large segments. Traffic was counted as it crossed the screen lines and then compared with the traffic estimates obtained from the model. As indicated in Table 4, the screen-line checks were usually within 10% of the actual traffic counts. Further, similar checks were carried out for mass transit and the results indicate a comparable degree of accuracy.

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[5] "Forecasting Peak Hour of Travel," by Alan M. Voorhees, Bulletin 203, Highway Research Bd., Washington, D. C., 1958.

In addition to the screen-line checks, information was amassed on employees' residences in several industrial plants. This information was checked against estimates developed by the gravity model. As indicated in Table 5, the technique accurately portrays the proportion of trips within specific travel times of the employment area. In making this comparison on a zone-to-zone basis there was a greater deviation between the actual and theoretical estimate. For example, when zone-to-zone volumes were 100, the root-mean square error was approximately 50%; for volumes of 1,000, the error was about 20%; for volumes of 10,000, the error was about 10%. (Root-mean square error means that two thirds of the time this error will be less than specified.) This series of checks indicated that, statistically speaking, the Baltimore traffic model was about as accurate as a 5% home-interview study.

An interesting historical check was based on data from 1926, 1946, and 1958 studies that revealed home-work relationships. For this time span the gravity model was suitable if the appropriate travel time for each era was used, an especially extraordinary finding because the average work trips had become 40% longer and travel times had changed drastically over the years.

PROJECTIONS

In projecting future travel, it should be recognized that traffic patterns depend upon the transportation alternatives that are offered to the public and the land use patterns that develop. Hence, it may be necessary to make several projections. In Baltimore, two were completed. The first was based upon a plan that called for extensive highway improvement with few transit changes; the second was derived from a plan that entailed several rapid transit lines and the completion of only the interstate highway systems.

TABLE 6.—CEILING FOR CAR OWNERSHIP PER HOUSEHOLD

Residence Type	Autos Per Household
Single Family	
new area	1.6
old area	1.0
Two Family	
new area	1.2
old area	0.9
Row House	
good transit and poor parking	0.4
good transit and good parking	0.6
poor transit and good parking	1.0
Elevator Apartments	
good transit and poor parking	0.2
good transit and good parking	0.4
poor transit and good parking	0.6

For the first plan, the traffic projections were based on the Baltimore Regional Planning Council's forecast of population and employment distribution. Car-ownership forecasting was implemented in several ways for comparative purposes. The method finally selected was based on a BPR study. It showed that income of household and type of residential area had a direct bearing on the number of cars per household. The study also revealed an increase in car ownership for specific residential areas until the income level reached a range of from $8,000 to $10,000 per year. Beyond this range car ownership leveled off. This means that, in effect, there was a ceiling for car ownership for the various types of residential areas (see Table 6). The number of cars that would be garaged in each residential zone was estimated on the basis of trends of existing car-ownership patterns and anticipated income levels for various zones.

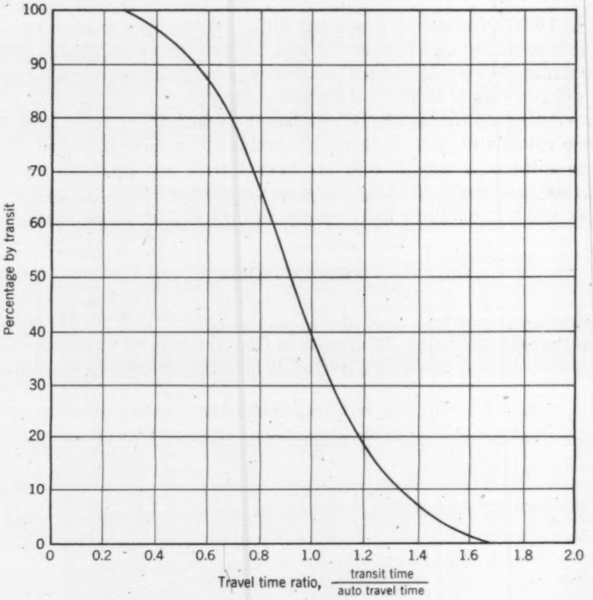
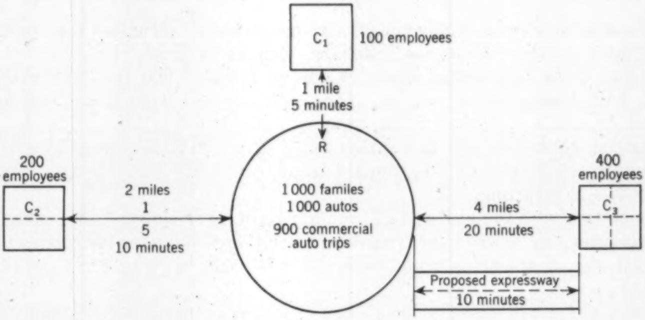


FIG. 2.—TRANSIT ASSIGNMENT CURVE

The existing travel times between zones were not used in projecting travel for the first plan: Travel times resulting from the development of an extensive freeway system were used instead. This was done to reflect the fact that improved highway facilities tend to increase travel length, as revealed by the historical checks previously mentioned. To sum up, the traffic forecasts measured the effects of anticipated increases in population and employment, car ownership, and expected increases in auto speeds.

The projection of traffic in the transit plan was accomplished on a somewhat similar basis. Car-ownership patterns in the vicinity of proposed rapid transit lines were adjusted in accordance with Table 6. The auto-travel times between zones reflected a more limited freeway system. Certain changes were made in the land use forecasts, specifically, a 20% employment increase in the CBD.


C_1
100 employees
1 mile
5 minutes
R
200 employees
C_2
2 miles
1
5
10 minutes
1 000 familes
1 000 autos
900 commercial auto trips
400 employees
C_3
4 miles
20 minutes
Proposed expressway
10 minutes

"PULL" (1)	PERCENTAGE OF TOTAL "PULL" (2)	NO. OF TRIPS (3)	VEHICLE MILES (4)
a) Existing			
FROM C_1 = 100 x 4.0 = 400	57	456	456
FROM C_2 = 200 x 1.00 = 200	29	232	464
FROM C_3 = 400 x 0.25 = 100	14	112	448
TOTAL "PULL" 700	100%	800	1368 MILES
b) After Expressway is Built			
FROM C_1 = 100 x 4.00 = 400	40	320	320
FROM C_2 = 200 x 1.00 = 200	20	160	320
FROM C_3 = 400 x 1.00 = 400	40	320	1280
TOTAL "PULL" 1000	100%	800	1920 MILES

FIG. 3.—EXAMPLE OF USE OF GRAVITY MODEL

A new set of auto and transit patterns was formulated by using these criteria. But, it was recognized that a certain portion of the population would probably shift from auto to transit travel in the event rapid transit lines became a reality. The estimate of the volume of this shift to rapid transit was calculated using the curve in Fig. 2. The curve was applied to only 75% of auto trips, that percentage of trips for which autos were not necessary. It was possible to forecast the traffic and auto patterns for the second plan by completion of the latter procedure.

EXAMPLE

To assist in understanding the gravity model and to visualize it as employed to estimate traffic volume, the following example is given.

In Fig. 3 the residential area designated R had 1,000 families within its limits, each family having one car. Three commercial areas are in the vicinity C1, that is a mile distant or 5 min away by auto. It has a total of 100 employed in retailing activities. C2 is 2 miles away, or about 10 min away by car, with 200 retail employees. C3 is 4 miles away, or 20 min away by car, with 400 employees in retailing.

Consistent with the analysis of commercial trip frequency, this means that 800 trips per day would start from residential area R. Based on computations shown in Fig. 3, 456 trips would be made to C1; 232 trips to C2; and 112 trips to C3.

Consider for a moment an imaginary expressway that would enable residents to travel to C3 in half the time, or 10 min. From the computations shown, 800 trips would be reoriented in the following manner: 320 to C1, 160 to C2 and 320 to C3 resulting in 558 more vehicle miles or an increase of approximately 40%. Further, the expressway would accommodate 208 additional vehicles and would increase traffic by nearly 185%.

Similar reorientation in traffic movement would be observed if another type of land use were established in the vicinity, for instance, a new shopping center.

BENEFITS

As in a conventional type of home-interview O-D study, a synthetic study of this type permits the analysis of several transportation alternatives. However the use of a model allows greater flexibility and a better opportunity to evaluate these alternatives.

By studying the two alternatives in Baltimore, the role of mass transit transportation was clearly defined. This survey revealed that Baltimore transit services, no matter how extensive, cannot be considered a substitute for highway improvements. Nor will they drastically reduce highway building requirements.

The application of the traffic model in Baltimore, has provided the planning staff with a clearer conception of the city's traffic problems and, in addition, has helped it to envision the effect that land use arrangements have on traffic patterns. Factually, any type of land-use plan can be evaluated with such a model, and it is possible to investigate many transportation alternatives, and to decide on the one making most "transportation sense."

Traffic models have been used effectively in other cities, for example, the Washington, D. C. Mass Transit Study conducted by the National Capital Regional Planning Commission evaluated one land use plan and four alternative transportation plans. Each alternative plan was created with the objective of meeting the transportation requirements of the proposed land-use plan. Each plan varied with the type of transit facilities and service to be provided, as well as with the numerous highway improvements to be incorporated. Traffic models like the one developed for Baltimore, were used to estimate expected patronage of various transit proposals, in addition to highway volumes. In a comparison, the estimates presented a picture of the advantages of each of the four alternatives. With this type of data, Washington is now deciding on its transportation program.

The city of New Haven recently completed a study based on another method of synthesis from land-use factors. Conducted over a period of a few months, this study provided the city planners and traffic engineers with a method of determining the advantages of different interstate locations. Similar studies are under way now in Hartford and Boston, Massachusetts. In Boston, they are undertaking about 1,000 interviews in order to help strengthen the model and to make it adaptable to any local conditions to which it must be adapted.

This type of approach, that of using a model with a limited amount of interviewing, makes a great deal of sense in that local information is obtained that strengthens the model. At the same time, it is possible to adjust it to local conditions. Though it is possible to carry out a synthetic origin and destination study without interviews it is very helpful to conduct a limited number to check the various factors used.

SUMMARY

For more comprehensive highway planning, traffic models offer a variety of advantages. The factors influencing traffic patterns are better understood as a result of their use, and a more sound factual basis is established from which to base conclusions. Mathematical models also permit a more thorough evaluation and testing of alternatives, which results in more realistic overall plans.

Traffic models hold much promise because of their economy. The relatively low cost, (Baltimore's model approximated $25,000) the limited staff necessary, and the technically simple procedures that are involved undoubtedly will contribute to their increasing popularity.

The benefits accruing to professionals in experimenting with and using traffic models should urge us to improve applicable techniques. When this is done, and a more effective mathematical traffic model is the outcome, urban-highway planning will be more exacting.

Engineers have the responsibility of expediting improvements to the traffic model, because in the years ahead it will be extremely valuable for the entire profession.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 3233

PREDICTION OF MAXIMUM PRACTICAL BERTH OCCUPANCY

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With Discussion by Messrs. Donald C. Hill; and Thomas J. Fratar, Alvin S. Goodman, and Austin E. Brant, Jr.

SYNOPSIS

Studies of observed berth occupancy patterns at commercial marine terminals have been made for many United States ports. Based on the close agreement between these observed patterns and theoretical frequency distribution functions, a method for predicting maximum berth occupancy has been developed.

TYPES OF MARINE TERMINALS STUDIED

Marine terminals may be divided into two groups, based on type of operations: commercial terminals, which are operated more or less on a public basis for the use of several shippers or consignees and which handle the vessels of several shipping companies; and proprietary terminals, which are operated on a private basis for the use of a single shipper or consignee or which handle the vessels of a single shipping company or of several companies which coordinate their vessel schedules.

The occupancy of a proprietary terminal is controlled by the terminal operator, which may be a shipper, consignee, or shipping company, who can adjust vessel arrivals and departures to attain as high a berth occupancy as re-

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quired by commerce demands. The occupancy of a commercial terminal, however, is determined largely by ship arrivals and departures over which the terminal operator has little or no control. Moreover, vessel arrivals and departures are influenced and often disrupted by one or more of a multitude of unpredictable factors such as weather, ice conditions, operating difficulties, loading and unloading problems, fluctuations in trade, holidays, labor strikes, and the like. Thus, while ship schedules might indicate a potentially high berth occupancy at a commercial terminal, the actual berth occupancy will often be entirely different.

INFLUENCE OF BERTH OCCUPANCY ON THE CAPACITY OF COMMERCIAL MARINE TERMINALS

Operations at a commercial marine terminal may be divided into three distinct and independent functions:

1. The movement of cargo into or out of rail cars, trucks or barges;
2. The transit storage of cargo at the terminal; and
3. The movement of cargo into or out of vessels.

The practical operating capacity of a terminal is the volume of cargo which can be handled through the terminal with reasonable efficiency and only infrequent congestion, and may be limited by any one of the terminal's three operating functions.

At many United States ports, the factor determining the practical operating capacity of a commercial marine terminal is the capacity for moving cargo into or out of vessels berthed at the terminal.

This capacity is basically the product of two components; the cargo handling rate (expressed in tons per day of occupancy per berth) which can reasonably be attained and the number of days in a year that the berths at the terminal can be occupied under the prevailing operating conditions without undue congestion. This latter component of terminal capacity is the subject of this paper.

The maximum practical occupancy of a group of berths at a commercial terminal is largely limited by the ability of the group to meet the demands imposed by the pattern of ship arrivals and departures. In a hypothetical case, where a terminal contains a very large number of berths and the level of business is also very high, it can be expected that each berth will be reoccupied very quickly each time it is vacated. The occupancy per berth under these conditions would be very high. At the other extreme, where a terminal has only one or two berths, ship arrivals cannot be matched nearly as well with ship departures. Since the use which can be made of each berth is greater at terminals having a large number of berths, the operating capacity of a terminal increases at a greater rate than the proportional increase in the number of berths.

OBSERVED BERTH OCCUPANCY PATTERNS

Two sources of data were used for the analyses presented herein; first, data on berth occupancy at general cargo marine terminals collected in the course of port development studies at Houston, Texas,[4] Portland, Oregon,[5] and Baltimore, Maryland;[6] and second, a study of the arrivals and departures of general cargo vessels during a six-month period at the ports of Boston, New York, Philadelphia, Baltimore, and Hampton Roads prepared by the Baltimore Association of Commerce.[7] The Baltimore Association of Commerce data were analyzed to determine the number of vessels in each port on each day of the period studied. No analyses were made for the port of New York because of the special conditions of terminal operation at that port, where many of the larger shipping companies operate terminals for the exclusive use of their own vessels. No analyses were made for the port of Baltimore since more detailed data were available elsewhere.[6]

A typical berth occupancy pattern derived from these data is given in Fig. 1, which shows the number of general cargo vessels in Philadelphia, Pa., on each day of July, 1955. During the period July through December, 1955, the number of general cargo vessels in Philadelphia ranged from 2 to 25. The randomness of berth occupancy is apparent.

The pattern of occupancy of commercial general cargo berths at a port can be more clearly shown by a frequency distribution curve. Fig. 2 shows the frequency distribution of the number of general cargo vessels in the port of Boston, Mass., during the period July through December, 1955. For example, on 4 days in this 184-day period, no vessels were in Boston; on 10 days, 1 vessel was in port; on 20 days, 2 vessels were in port; and so forth.

Similar studies at other United States ports indicate that the patterns of berth occupancy for commercial terminals are essentially random in that the following conditions are met:

1. The probability of a number of vessels in a port on a given day is not influenced by the number of vessels in the port on any other day; and
2. Each vessel arrives and departs independently of the movements of other vessels.

These conditions represent fairly well the operations at commercial general cargo terminals at the ports studied. When these conditions are met, the berth occupancy patterns can be represented by a theoretical probability distribution function.

STATISTICAL ANALYSIS OF BERTH OCCUPANCY PATTERNS

To determine the probability distribution functions that most nearly fit the observed data, a detailed statistical analysis was made.[8] It was found that only two of the theoretical probability distribution functions would meet the conditions of asymmetry and flatness imposed by the observed berth occupancy patterns. These two theoretical functions, the Poisson Law and the Binomial Law, were then examined further to determine which one had a higher level of agreement with the observed data. It was found that the Poisson Law, which can be derived directly from the conditions of randomness stated above, is more suitable than the Binomial Law to represent the occupancy of berths at commercial marine terminals. Using a different method of derivation, moreover, the Poisson Law can be shown to be a limiting case of the Binomial Law.

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[4] "Master Plan Survey for Port of Houston," prepared for Bd. for Commrs., Harris County, Houston Ship Channel Navigation Dist., Tippetts-Abbett-McCarthy-Stratton, 1953.

[5] "Portland Harbor Development Survey," prepared for the Comm. of Pub. Docks, Portland, Oreg., Tippetts-Abbett-McCarthy-Stratton, 1959.

[6] "The Port of Baltimore," prepared for the Maryland Legislative Council Port Committee, Tippetts-Abbett-McCarthy-Stratton, 1954.

[7] "Service Provided at the Ports of Boston, New York, Philadelphia, Baltimore and Hampton Roads by Vessels Carrying General Cargo in Foreign Trade which Entered or Cleared During the Period July - December, 1955," submitted before the Interstate Commerce Comm., Investigation and Suspension Docket No. 6615, W. S. Hamill, Baltimore Assn. of Commerce, 1957.

[8] "Probability and Its Engineering Uses," by Thornton C. Fry, D. Van Nostrand Co., New York, 1928.

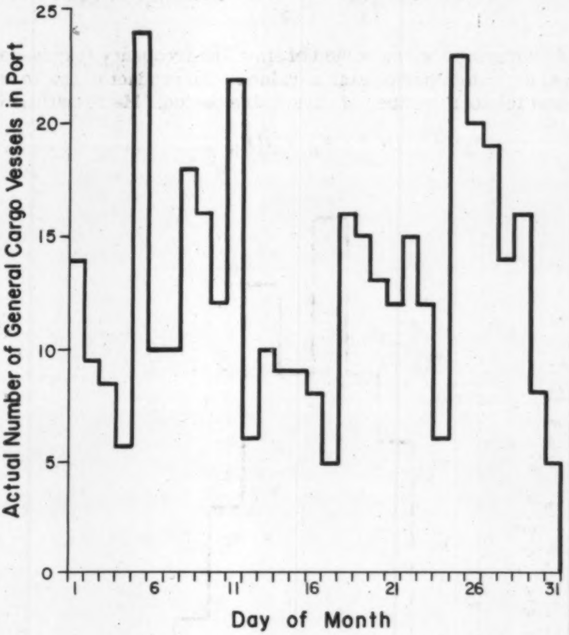

FIG. 1.—BERTH OCCUPANCY PATTERN

The Poisson Law is

$$P_n = \frac{(\overline{n})^n \; e^{-\overline{n}}}{n!} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which P_n is the probability of n berths being occupied at a terminal (or n vessels being in a port) on a given day, n represents the number of berths occupied (or the number of vessels in port), the Napierian-logarithm base e is 2.71828. . . , n! represents n factorial = n (n-1) (n-2) . . . (3) (2) (1), and $\overline{n}$ is the average number of berths occupied (or average number of vessels in port). The Poisson Law has two important properties: first, the summation for a given value of $\overline{n}$ of the values of P_n as n varies from 0 to infinity is unity; and second, the summation for a given value of $\overline{n}$ of the products of n and

P_n as n varies from 0 to infinity is $\bar{n}$. These properties are expressed as:

$$\sum_{n=0}^{\infty} P_n = 1 \quad \dots\dots\dots\dots\dots\dots \quad (2)$$

and

$$(\bar{n}) = \sum_{n=0}^{\infty} (n) \left(P_n\right) \dots\dots\dots\dots\dots\dots \quad (3)$$

For a study covering a given period of time, the frequency (expressed in number of days) of occurrence of each n-value is the product of the corresponding P_n-value and the total number of days in the period. Mathematically,

$$F_n = N\left(P_n\right) \dots\dots\dots\dots\dots\dots \quad (4)$$

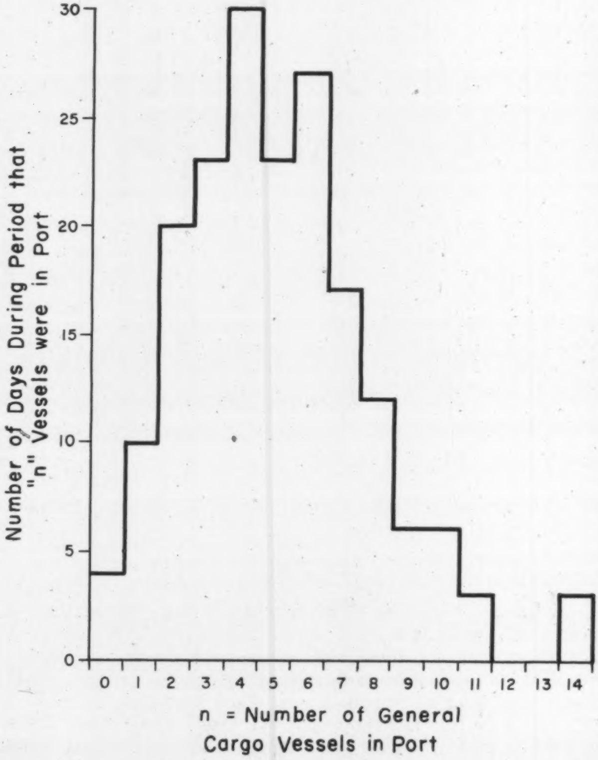

FIG. 2.—FREQUENCY DISTRIBUTION CURVE

in which N is the total number of days in the period, and F_n is the theoretical number of days in the period that n berths are occupied at a terminal (or n vessels are in a port), as determined from the Poisson Law.

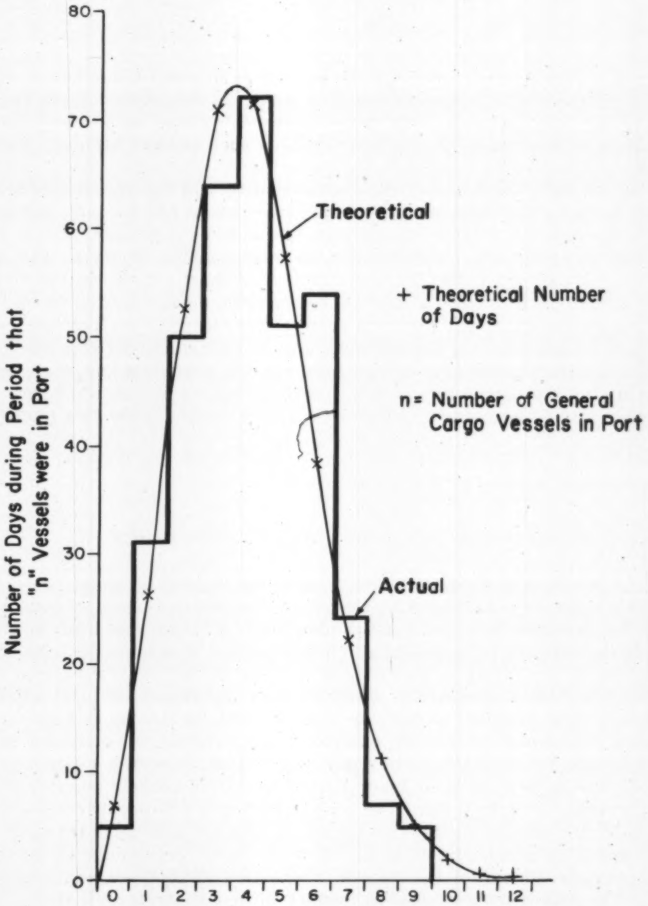

FIG. 3.—ACTUAL AND THEORETICAL FREQUENCY DISTRIBUITON OF GENERAL CARGO VESSELS IN PORT

Unfortunately, because of the nature of probability analysis, the applicability of a probability distribution function to a given set of observed data cannot definitely be determined. The probability that the observed data could result from the Poisson Law ("goodness of fit") can be determined, however, by use of the chi-square test. The value of chi-square is determined as

$$\chi^2 = \sum_{n=0}^{\infty} \frac{\left(F_n - f_n\right)^2}{F_n} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(5)}$$

in which f_n is the actual number of days in the period that n berths are occupied at a terminal (or n vessels are in a port), as determined from the observed data. The goodness of fit between the calculated values and the observed data is determined by the use of tables such as have been given elsewhere.[8]

The goodness of fit between the observed data and the theoretical values obtained by using the Poisson Law ranges from about 65% (considered to be excellent agreement) to less than 1% (considered to be poor agreement). Poor agreement was generally found when the average number of berths was large and the period for which data were available was short. It is believed that if data for longer periods were available, better agreement would be found.

The levels of agreement obtained from these analyses are believed to be reasonable enough that the Poisson Law can be considered as representative of berth occupancy patterns at commercial marine terminals and, thus, can be used to predict the maximum practical berth occupancy of such terminals. An illustration of this agreement is shown on Fig. 3, which presents a histogram of the observed data and a curve of the theoretical values for the year 1957, at Portland, Oreg. Observed data and theorectical values for several ports and terminals are compared in Table 1 and Table 2.

MAXIMUM PRACTICAL BERTH OCCUPANCY

In determining the capacity of a commercial marine terminal, a level of congestion must be established to represent the practical limit beyond which the service provided to vessels is not adequate. A reasonable limit for berth occupancy is believed to be that at which the number of vessels desiring to occupy berths at the terminal exceeds the number of berths available on about 5% of the time, or an average of about 18 days per year. On these 18 days, vessels would be required to wait for a berth to be vacated by another vessel. It is found that this criterion for maximum practical berth occupancy is met when an average of 0.35 vessel per day requires berthing at a 1-berth terminal; 0.82 vessel per day at a 2-berth terminal; 1.36 vessels per day at a 3-berth terminal; and so forth. The corresponding berth occupancies are 29% of the calendar days for a 1-berth terminal, 38% for a 2-berth terminal, 43% for a 3-berth terminal, and so forth. A curve of the percentage of berth occupancy versus number of berths with congestion 5% of the time is shown on Fig. 4. The maximum practical occupancy increases from 29% at a 1-berth terminal to 66% at a 15-berth terminal and would continue to increase (at a diminishing rate) for greater numbers of berths.

TABLE 1.—COMPARISON OF ACTUAL AND THEORETICAL NUMBER OF VESSELS AT COMMERCIAL GENERAL CARGO TERMINAL

(n) Number of General Cargo Vessels at Terminal on a Given Day	Number of Days During Period n Vessels at Terminal		Average Number of Vessels at Terminal	Value of chi-square	Goodness of Fit
	(f_n) Actual	(f_n) Theoretical From Poisson Law			
(1)	(2)	(3)	(4)	(5)	(6)
(a) Portland, Oregon: January-December 1957					
0	5	6.6	4.02	11.97	16%
1	31	26.3			
2	50	52.9			
3	64	71.0			
4	74	71.3			
5	51	57.3			
6	54	38.4			
7	24	22.1			
8	7	11.1			
9	5	5.0			
10	0	2.0			
11	0	0.7			
12	0	0.3			
TOTALS	365	365.0			
(b) Houston, Texas: January-December 1952					
0	3	5.2	4.25	9.27	32%
1	26	22.2			
2	45	47.2			
3	56	66.8			
4	78	71.0			
5	71	60.3			
6	36	42.7			
7	27	25.9			
8	17	13.8			
9	6	6.5			
10	1	2.8			
11	0	1.1			
12	0	0.5			
TOTALS	366	366.0			
(c) Baltimore, Maryland: January, March, April, June, July, and September 1959					
0	7	3.9	3.85	3.97	65%
1	13	15.0			
2	27	28.9			
3	39	37.0			
4	32	35.6			
5	23	27.4			
6	24	17.6			
7	11	9.7			
8	5	4.7			
9	2	2.0			
10	0	0.8			
11	0	0.3			
12	0	0.1			
TOTALS	183	183.0			
(d) Baltimore, Maryland: January-September 1952					
0	13	19.2	2.65	29.03	Less than 1%
1	44	50.7			
2	74	67.2			
3	57	59.4			
4	60	39.3			
5	23	20.8			
6	0	9.2			
7	0	3.5			
8	0	1.2			
9	0	0.5			
TOTALS	271	271.0			
(e) Baltimore, Maryland: January, March, April, June, July, and September 1959					
0	21	18.2	2.31	8.83	12%
1	47	42.0			
2	44	48.4			
3	35	37.3			
4	15	21.5			
5	10	10.0			
6	5	3.8			
7	6	1.3			
8	0	0.4			
9	0	0.1			
TOTALS	183	183.0			

TABLE 2.—COMPARISON OF ACTUAL AND THEORETICAL NUMBER OF GENERAL CARGO VESSELS IN PORT

(n) Number of General Cargo Vessels in Port on a Given Day (1)	Number of Days During Period n Vessels in Port		Average Number of Vessels in Port (4)	Value of chi-square (5)	Goodness of Fit (6)
	(f_n) Actual (2)	(f_n) Theoretical From Poisson Law (3)			
(a) Boston, Massachusetts: July-December 1955					
0	4	1.3	5.00	17.03	3%
1	10	6.2			
2	20	15.5			
3	23	25.8			
4	30	32.3			
5	23	32.3			
6	27	26.9			
7	17	19.2			
8	12	12.0			
9	6	6.7			
10	6	3.3			
11	3	1.5			
12	0	0.6			
13	0	0.3			
14	3	0.1			
TOTALS	184	184.0			
(b) Philadelphia, Pennsylvania: July-December 1955					
0	0	0	12.25	72.07	Less than 1%
1	0	0.1			
2	2	0.3			
3	1	0.8			
4	3	2.0			
5	8	4.1			
6	10	7.2			
7	4	11.1			
8	14	15.1			
9	17	18.5			
10	14	20.5			
11	9	21.0			
12	19	19.8			
13	13	17.3			
14	16	14.1			
15	10	10.8			
16	10	7.8			
17	7	5.3			
18	6	3.4			
19	6	2.1			
20	3	1.2			
21	1	0.7			
22	6	0.4			
23	3	0.2			
24	1	0.1			
25	1	0.1			
TOTALS	184	184.0			
(c) Hampton Roads, Virginia: July-December 1955					
0	1	0	11.03	68.55	Less than 1%
1	7	0			
2	3	0.2			
3	2	0.7			
4	2	1.8			
5	8	4.1			
6	7	7.5			
7	9	11.8			
8	17	16.2			
9	9	19.9			
10	20	21.9			
11	17	22.0			
12	12	20.2			
13	14	17.1			
14	16	13.5			
15	9	9.9			
16	7	6.8			
17	7	4.4			
18	6	2.7			
19	3	1.6			
20	2	0.9			
21	4	0.5			
22	1	0.2			
23	0	0.1			
24	0	0			
25	0	0			
26	0	0			
27	1	0			
TOTALS	184	184.0			

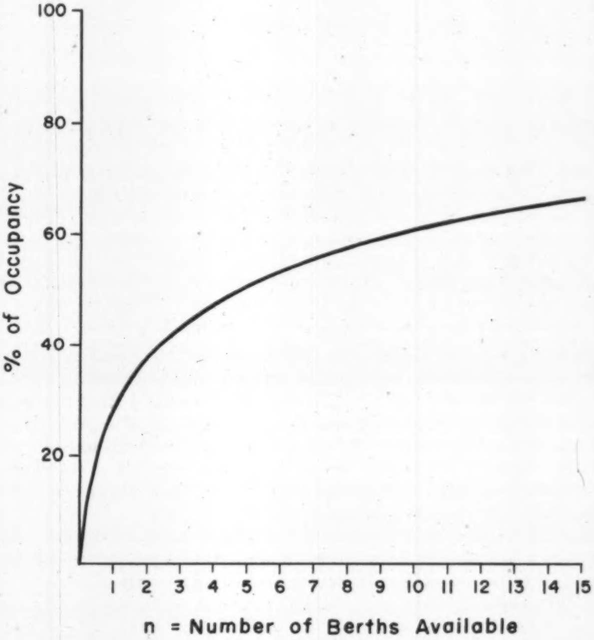

FIG. 4.—PERCENTAGE OF OCCUPANCY WHICH CAN BE ATTAINED WITH FROM 1 TO 15 BERTHS

CONCLUSIONS

The Poisson Law of Random Distribution appears to agree reasonably well with observed data on berth occupancy at commercial marine terminals. Moreover, this law is the basis for the solution of many problems of the quantity of equipment needed to perform a given service when the demands for that service are random. Because of the generally good agreement between observed data and the theoretical values given by the law, it is believed that it is reasonable to use the Poisson Law to determine maximum practical berth occupancies at commercial marine terminals. In so using the law, the criterion of congestion which has been set forth herein is that the number of vessels desiring to occupy berths at the terminal is greater than the number of berths available for about 5% of the time, or that the terminal is congested on only about 18 days per year. With this criterion, it is found that the maximum practical berth occupancy at a commercial marine terminal ranges from 29% for a 1-berth terminal to 66% at a 15-berth terminal.

DISCUSSION

DONALD C. HILL.[9]—In the plans for development of general cargo terminal facilities at Hampton Roads, Virginia, the problem of financing is paramount. The Virginia State Ports Authority can finance the construction of new facilities only if they are able to demonstrate, to the satisfaction of the presumptive tenant, that the cargo potential to the terminal is of sufficient volume to justify the rental demanded. Obviously, then, it becomes essential that they be able to predict with a reasonable degree of accuracy the expected berth occupancy of the facility.

Messrs. Fratar, Goodman, and Brant base their prediction formula on the Poisson Law of Random Distribution. The Poisson Law, however, as would be true of any law of probability, is applicable only when all controllable factors have been eliminated, and the law applied only to those factors that may be characterized as pure chance. It follows, therefore, that the Poisson Law can be used for the prediction of berth occupancy only if applied to a terminal in which all available berths are equally attractive to all users. This law is not applicable unless all berths within the terminal under consideration are equally modern, equally maintained, and with equal access to all forms of transportation on both land and water sides.

At Hampton Roads specifically, the Poisson Law will give a "goodness of fit" of a very low order if we attempt to apply this formula to the entire general cargo terminal complex. It would be applicable only if applied to a single group of terminals served by a single carrier, such as the Chesapeake and Ohio facilities at Newport News, or the Norfolk and Western facilities at Lamberts Point in Norfolk, Virginia, or the Atlantic Coast Line facilities at Pinners Point in Portsmouth, Virginia. Even then, an unknown constant should be introduced to compensate for the variable age of the individual piers within the group.

THOMAS J. FRATAR,[10] F. ASCE, ALVIN S. GOODMAN,[11] M. ASCE, and AUSTIN E. BRANT, JR.,[12] A. M. ASCE.—Mr. Hill's thoughful discussion of the application of probability theory to berth occupancy at Hampton Roads brings out some important concepts that are implicit in the theory.

Mr. Hill notes that the Poisson Law applies only when the assumption can be made that all the available vessel berths in a group of berths are equally attractive to all shippers. This is certainly true where vessel berths are leased on an individual basis to a terminal operator (that may be a shipping company, a stevedoring company, a railroad, or the like), because the operator is interested principally in the efficiency and operating capacity of his berth. Under this condition, the attractiveness of a vessel berth is measured by the cost to the user, or the cost of handling cargo at the berth.

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At a port in which all vessel berths are actually operated by a port-wide agency, the cost to the user of a terminal will probably be the same regardless of the efficiency and operating capacity of individual berths. Under this condition, if all berths in a port are located within the same general area and have equal accessibility to railroads and highways serving all parts of the port's tributary area, the operational characteristics of all berths need not be the same to obtain maximum efficiency in utilization. In other words, the most efficient berth in a group of berths could be operationally scheduled to handle a much greater proportion of the total commerce of the group than the least efficient berth. Considering the group as a whole, however, the over-all performance could be predicted by the Poisson Law.

With specific application to Hampton Roads, where berths are located in various sections of the port and have considerably different inland transport connections, terminals in all sections of the port must be made equally efficient to be equally attractive to shippers. This illustrates the advantage of imporving the least efficient berths to the level of the more efficient ones as the first step in a portwide development program.

With respect to Mr. Hill's comment that Newport News and Norfolk should be considered as separate ports, it should be noted that the application of the Poisson Law shows that two independent sets of berths have a much lower operating capacity than the combined group. For example, two groups of six berths each have a maximum practical occupancy of 53%, whereas a single group of twelve berths has a maximum practical occupancy of 63%. This increased efficiency can be attained by consolidating operational control of all berths in a port in a single agency and by providing as far as possible equal accessibility for all terminals.

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TRANSACTIONS

Paper No. 3237

APPROACH TO THE STUDY OF OVERSEAS TRAFFIC POTENTIALS

By Wilfred G. McLennan,[1] Aff., ASCE

SYNOPSIS

The methodology for estimating traffic potentials of general cargo in the United States foreign trade are examined herein. Comparisons are made of existing commerce into and out of the Great Lakes tributary area through available commerce routes of the past and new commerce routes resulting from the opening of the St. Lawrence Seaway.

INTRODUCTION

A methodology for estimating traffic potentials of general cargo in United States foreign trade will be analyzed herein. Such methodology can be used in measuring the impact of deep-draft navigation on overseas commerce into and out of the Great Lakes area through the connecting channels on the Great Lakes and the St. Lawrence Seaway. Based on a nation-wide study of the origin of exports and the terminus of imports in the Unites States, these methods may be the key to greater knowledge of transportation economics in foreign trade.

Therefore, an explanation, in general terms, of basic problems and possible solutions will be presented. In so doing it is pointed out that any formulas or methodology proposed herein, not yet accepted by the United States Army Corps of Engineers, represent the views of the author and not necessarily those of the Corps.

One particular aspect of the study will be emphasized above others to illustrate methodology for measuring the volume of general cargo tonnage expected to flow through the ports on the Great Lakes via connecting channels and the

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St. Lawrence Seaway. With respect to such general-commodity overseas traffic, the main consideration will center around means of gaging, with reasonable accuracy, the nature and potentials of the export traffic originating and import traffic terminating in an area tributary to the ports on the Great Lakes.

To place the problem of estimating Great Lakes-overseas traffic potentials in proper perspective it is first necessary to indicate the reason for the United States Army Corps of Engineers undertaking the study of potential overseas traffic through Great Lakes ports, and second, why information on traffic potentials is important, whether considering future traffic through the Great Lakes ports, the connecting channels and the Seaway, or through seaboard ports on the Atlantic, Gulf, and Pacific coasts of the United States.

Authority for the Traffic Study.—A Great Lakes Harbors Study was authorized by resolutions at the Committee on Public Works, United States Senate, adopted on May 18, 1956 and by the Committee on Public Works, House of Representatives, United States, adopted on June 27, 1956. These resolutions requested the Board of Engineers for Rivers and Harbors to review the reports of the Chief of Engineers on the Great Lakes connecting channels.

The advisability of further improvements of the harbors on the Great Lakes in the interest of present and prospective deep-draft commerce were to be determined with due regard to the scheduled time of completion of the St. Lawrence Seaway and the connecting channels between the Great Lakes.

Preparation of such a comprehensive study includes an evaluation of potential overseas traffic through the Great Lakes ports. The methodology of applicable research will be examined.

The Six Phases of the Traffic Study.—Basic procedures of a Great Lakes-overseas traffic study suggest an arrangement of its several major phases so that development of the methodology can be more readily followed.

Six such phases of a traffic study of this nature are as follows:

1. A general approach to the study that involves the development of methodology and mechanics of determining points of origin and destination of commodities in United States foreign trade;
2. The origin-destination phase, that involves an analysis of points at which exports and imports enter the flow of United States overseas foreign trade;
3. The base period phase (1955-1956), that involves a study of United States production, consumption, exports and imports past and present to establish base from which to estimate future traffic;
4. The projection phase (1960-2010), that involves a consideration of economic growth prospects, including foreign trading on a commodity basis, for United States in general and Great Lakes tributary area in particular, in order to estimate potential traffic over the 50-year period, 1960-2010;
5. The transportation analysis phase (1958), that involves a study of overland and ocean operating costs and freight rates to establish cost and rate differentials for routings via Great Lakes ports versus routings via seaboard ports;
6. The potential great lakes traffic phase (1960-2010), that involves an evaluation of factors considered in the previous phases so as to indicate level of future traffic expected to be generated in the Great Lakes tributary area and oriented toward the Great Lakes, and allocation of that potential traffic to individual harbors on the Great Lakes.

Each of these phases provides information by itself, but it is necessary to use a combined research effort in estimating Great Lakes and Seaway traffic potentials.

GENERAL APPROACH TO THE STUDY

During recent years many estimates of the foreign overseas traffic potentials of the Great Lakes and the St. Lawrence Seaway have been made. All, of necessity, has to be made without benefit of information, now available (1958), as to the origin of exports or the terminus of imports within the United States.

Defining the Great Lakes Area.—The Great Lakes tributary area can be defined in several ways, all varying as to the sphere of influence extending into the hinterland. Of prime importance, of course, is the heavy concentration of traffic generated in the immediate vicinity of the port cities themselves. In the ever widening circles the hinterland areas of each port reach out until they merge. For present purposes, and until the final results of the traffic study show which routings are the most economical, an area that conforms roughly to the sphere of influence of the Great Lakes ports can be considered as including the western portions of New York and Pennsylvania, and most of the mid-west states such as Ohio, Indiana, Michigan, Illinois, Wisconsin, Minnesota, Iowa, Missouri, North Dakota, South Dakota, Nebraska, and Kansas.

This area, represents approximately one-quarter of the land area of continental United States and one-third of the total population. Within its boundaries is about one-third of the nation's standard metropolitan areas (a group of contiguous counties containing at least one city of 50,000 or more inhabitants). This area also accounts for more than one-third of the dollar value of the nation's manufacturing and almost one-half of the dollar value of its agriculture.

Most of the non-agricultural activity of the area is less than 200 miles from any port on the Great Lakes. Thus, until a final delineation of the Great Lakes tributary is determined, this region will serve as a reasonable approximation of the area tributary to the Great Lakes. Within the limits of the area tributary to the Great Lakes, however it is finally defined, exports begin their overseas journey and imports reach a destination in a factory, warehouse, store, shop, or home.

The delimitation of the Great Lakes tributary area will be determined largely through development of transportation differential data, that will indicate whether shipments to or from a particular point in the United States are oriented toward a Great Lakes port or toward a seaboard port. Transportation differentials can be figured on the basis of freight rates or carrier costs. Both of these bases were being considered in the Great Lakes Traffic Study.

Problems of the Traffic Study.—The problems in a study of this nature are generally threefold: First, how to determine where exports originate and where imports terminate within the Great Lakes tributary area; second, how to measure the magnitude of potential Great Lakes-Seaway foreign trade; and third, how to define the extent of the Great Lakes Tributary area. In the early consideration of these questions it became quite evident that no complete data were available on a nation-wide scale with respect to the following:

1. Location in continental United States of the points of origin and destination of United States overseas traffic;

2. Volume and commodity composition of traffic that originated or terminated at each inland location;
3. Ports through which Great Lakes area foreign trade entered or cleared the United States;
4. Distribution of traffic among inland carriers such as railroad, truck and barge lines;
5. Overseas areas of origin or destination of the foreign trade generated in the Great Lakes area; and
6. United States coastal ports through which most of the Great Lakes tributary area commodities are shipped.

With data now available and pertinent to these and many other questions, it was considered practicable to estimate traffic potentials of the Great Lakes area without having to use assumed points of origin and destination from which to calculate whether it costs less to ship via a Great Lakes port and the Seaway than overland through a Seaboard port.

THE ORIGIN-DESTINATION STUDY

Cooperative efforts of the Bureau of the Census, Department of Commerce (USBC) were most helpful in developing methods and procedures that, for brevity, may be referred to as the origin-destination study.

The origin-destination study is a most important key to many answers. It consists primarily of a sampling process applied to official records of United States foreign trade data together with questionnaires sent to each of 15,000 shippers to find out the actual point at which their export or import commodities entered United States foreign trade.

Sampling Techniques.—Because it was physically impossible from the standpoint of time and money to analyze all of the more than 2,000,000 export and import transactions that constitute the United States foreign trade each year, it was necessary to exclude certain categories of commodities (mostly bulk items) from consideration and to employ a sampling method for the remaining general cargo commodities. Sampling techniques developed by D. E. Church were designed to produce the greatest degree of accuracy for the largest volume of exports and imports to be studied. The size of the final sample was, of course, determined by the funds available.

Selection of Commodities.—The "universe" from which the sample was taken consisted of a deck of approximately 2,160,000 foreign trade (waterborne) punch cards containing data from import entries and export declarations filed for the calendar year 1956. The import deck contained about 660,000 cards and the export deck about 1,500,000 cards. "Special categories" and shipments below the weight and dollar cut-off for the USBC foreign trade tabulations were not included.

Preliminary to selecting a "gross sample" from the "universe" of export and import cards previously noted, various bulk commodities such as iron ore, limestone, coal, grains, certain minerals and bulk petroleum, for which special studies were in preparation, were removed from the basic "universe." The object was to obtain a sample representative of "linear-type" cargo rather than "tramp-type" cargo.

In devising procedures for drawing the gross sample it was assumed that any bulk commodity shipments remaining in the universe would move in large units, probably in excess of 100,000 lb. This assumption, upheld in subsequent tests, became the basis for a stratification system. The gross sample, for exports and imports separately, was selected from strata based on pounds. This stratification by weight of shipment and withdrawal rate for each strata showed the following data:

Size of Strata	Ratio of cards to be withdrawn from "universe"
Strata 1, - 1,000,000 lb + -	all cards
Strata 2, - 100,000 lb - 999,999 lb -	1 card in 10
Strata 3, - 10,000 lb - 99,999 lb -	1 card in 50
Strata 4, - under 10,000 lb -	1 card in 250

The gross sample withdrawal rate reduced the original universe deck from 2,160,000 cards to approximately 120,000 cards, half exports, half imports. While representing only 5% to 6% of the number of card transactions, the gross sample accounted for more than 90% of the total weight of shipments in the universe.

Prior to selection of the net sample, specific commodities in addition to those omitted from the "universe," were deleted from the gross sample also. These additional deletions from the export deck included petroleum (oils), cotton (unmanufactured), grains, soybeans and flaxseed, and sulphur. Petroleum (oils), bananas, sugar and pulpwood were deleted from the import deck. The final number of transactions in the net sample was approximately 7,500 export items and 7,500 import items. It represented about 15% of the number of card transactions in the gross sample and accounted for about 40% of the weight of all shipments in the gross sample.

The next step was to withdraw from Census Bureau files and to examine the respective documents from which each transaction in the net sample had been abstracted. Photostat copies were made from each original export declaration and import entry form, and the specific item selected in the sampling was marked in ink so as to indicate the particular commodity for which the shipper of the goods was to supply information.

As a result of the commodity selection and sampling technique used, it appeared feasible to conduct an origin-destination study with a total of only 15,000 observations. These 15,000 observations when expanded to the basic level of full export-import volume of foreign trade indicated a difference of only a few percentage points between tonnages represented by the sample estimate and in actual overseas traffic.

Preparing the Questionnaire.—Having completed the sampling process the next step was to prepare a questionnaire to accompany the photostated documents. In cooperation with the USBC a questionnaire form for submission to exporters and importers was designed and submitted to the Bureau of the Budget for approval. It contained questions to develop information as to, (a) how the item was acquired, for export or how it was disposed of when imported, (b) where acquired or distributed, and (c) by what means it was transported overland. It is noteworthy that of the 15,000 questionnaires in the Great Lakes Traffic Study mailed by the Bureau of the Census, more than 90% were completed and returned.

Processing Origin-Destination Data.—The overall view, as to the extent of processing necessary in the origin-destination study, can best be indicated by stating that at least twelve separate and identifiable decks of punch cards were necessary to record and correlate data pertaining to all elements of the 15,000 transactions.

The data received in reply to each questionnaire was transferred to punch cards and matched item by item with the Census Bureau's original foreign-trade cards so as to provide the primary origin-destination data. These data, in addition to the Census Bureau's reference items, include, for each specific commodity shipped, information regarding type of acquisition or disposition, type of transport (inland), commodity detail, weight strata, ports of lading and discharge, point of origin and destination in the United States, distance between United States ports and inland point of origin or destination, and weight in pounds, and dollar valuation, for each transaction sampled.

The foregoing procedures, after expansion, provided the basic data for use in the transportation analysis phase of the study by which it was possible to establish an area of potential advantage for shipments via the connecting channels and the Seaway and to make estimates of traffic allocations to individual harbors on the Great Lakes.

Integration of Origin-Destination Analysis with Other Phases of the Study.—The origin-destination study is but one phase of the Great Lakes traffic survey. It provides basic data from which other parts of the general traffic study are derived.

Specific carrier cost and freight rate data are incorporated at this point into these punch cards containing origin-destination data so as to make it possible to compute transportation differentials for each of eleven basic USBC commodity groups of import and export cargo moving over trade routes between the United States and twenty separate foreign areas. In a similar manner data are developed with respect to costs and rates for overland transportation, all of which is explained in the transportation analysis section of this paper.

BASE PERIOD PHASES (1955-1956)

The origin-destination study determines the various points in the United States at which exports and imports enter foreign trade in order to use such information to measure potential traffic expected to be generated in the Great Lakes tributary area.

The boundaries of the Great Lakes area were defined rather broadly. How to define those limits more exactly and how to estimate the potential traffic of that area is the purpose of the sections which follow.

Selection of the Base Year.—The first step in such a procedure requires that a base year be selected from which to project estimates into the future. For that purpose the two year period 1955-1956 was selected as a representative base.

The methodology involved in this portion of the Great Lakes traffic study is similar to the methodology that would be applied to a research problem in any number of fields. Beginning with the collection and organization of known facts or data, a framework of hypotheses or assumptions is established. From this base, the study proceeds toward the solutions of the basic problems of projection posed by the study.

In an economic study, the findings are not susceptible to "trial runs." The validity of the findings and conclusions of economic analysis must always await the passage of time for proof. However, the validity of the study depends a great deal upon the general acceptance of the known facts or data that stand up under the "rule of reason." It is the collection and organization of these factual data that comprise the base period phase of this traffic study.

What then constitutes the body of fact upon which this study rests? As already shown, the central problem of the study was the lack of official statistical information regarding origin or destination in the United States of commodities in the United States foreign trade, especially those likely to be affected by the deep draft ocean traffic that would be brought to the Great Lakes via the St. Lawrence Seaway. Thus, the base period can be used as a bench mark against which this future foreign trade flow of commodities into and out of the Great Lakes tributary region can be measured—commodity flows that must be identified with regard to four primary relationships:

1. Selection of the significant commodity flows to be studied;
2. identification of the origins of the exports and the destinations of the imports within the Great Lakes region;
3. determination of the relationship between the export commodities and the producing sectors of the national economy, and between imports and the areas of the United States in which consumed; and
4. determination of the routings by which the commodities presently move, overland and by sea in United States foreign trade.

Selection of Commodities.—In any study it is necessary to establish a system of priorities with regard to the number and detail in which the units are to be analyzed. One form of priority is to determine the relative importance of these commodities, basing the selection of each on the following factors:

1. Volume and significance of the commodity movement into or out of the Great Lakes area, in terms of both overall tonnage and of specific commodity groupings;
2. importance of the commodity to the economy of the Great Lakes area and to potential movement via the connecting channels and the St. Lawrence Seaway;
3. level of detail at which the analysis can properly be made, such as steel rolling-mill products as a whole, or separately for iron and steel bars, ingots, billets, and so on; and
4. adequacy of coverage achieved by the commodity selection in terms of total volumes in the Great Lakes tributary area and Seaway potential as well as with respect to interests of individual Great Lakes ports.

The selection of commodities finally included in a study of this nature requires a detailed analysis of several different tabulations of waterborne foreign commerce, covering a span of years. For the purpose of this report, the two year period 1955-1956 appears to be a representative base. In total some sixty export- and sixty import-commodity classifications were selected for special attention. In terms of tonnage these items averaged 88 million short tons annually in 1955 and 1956, or almost 90% of the volume of commodities to be covered in this traffic study. As mentioned previously, special bulk commodities, such as iron ore, limestone, coal, grains, certain minerals and bulk petroleum, that traditionally have accounted for almost all of the lake commerce, have been treated in special traffic studies.

Origins, Destinations, and Routings.—After the priorities for study have been established in terms of commodities, the next step is identification of these commodities in terms of their movements into and out of the Great Lakes region. First, the volume of the traffic flow for each commodity is studied as to specific origins of exports from the Great Lakes region and specific destination of imports into the area. Of importance also was the identification of the foreign areas for which exports from the Great Lakes were destined and the overseas area from which imports into the Lakes region originated. In addition, the particular route taken by the exported or imported commodity is considered in establishing the pattern of its movement through East coast, Gulf coast, or West coast ports of the United States. Thus, in this phase, the relative importance and significance of each commodity flow is established.

Information regarding commodity flows can be developed from several sources. The primary benefit is, of course, derived from the USBC origin-destination study, which, as already pointed out, provides, for the first time on a nation-wide scale, significant data regarding the commodity traffic flows previously mentioned. For those commodities with overseas traffic potentialities but which are not included in the USBC origin-destination study it is necessary to analyze foreign trade data statistics, domestic distribution of the commodities from Interstate Commerce Commission (ICC) waybill records, together with production and consumption data from USBC and other sources.

Producing-Consuming Relationships.—Statistics alone do not tell the whole story as to which sector of the nation consumes the imports or produces the exports. Analysis of these relationships can be developed from several sources.

1. A field interview program provides the basic core of information on important aspects of the base-period analysis in terms of commodity flows. Although confined to potential shippers within and near the Great Lakes port cities, for the most part, the breadth of coverage in terms of industry provides a representative cross section of commodity flows and the relationship of the commodities to basic producing-consuming sectors of the economy.
2. Staff analysis provides much of the basic data for relating foreign trade products to specific producing-consuming interests by comparing several different statistical codes, such as the standard industrial classification, the Department of Commerce export and import commodity classification, and the Association of American Railroads commodity classification that is used by the ICC for rail freight movements.
3. Contributions to the methodology of economic research in commodity traffic flows can also be derived from the use of USBC data on manufactures, Department of Agriculture publications, and the Bureau of Mines reports.
4. A special study of selected minerals, not included in the USBC origin-destination study, developed data that appears to be highly significant in terms of potential overseas traffic to the Great Lakes area.

Thus, the base period phase sets the stage for the study by selecting the commodities, describing their movements into and out of the Great Lakes region, and identifying them in terms of domestic geographic areas of origin or destination within the economic sectors of production and consumption of the United States.

Finally, in terms of volume, movements to and from specific foreign areas are identified by the patterns of traffic by land and by sea. The identification of the points of origin and destination and the routes between the domestic points and the foreign areas provides the basic framework for a transportation cost or rate analysis or both which will be discussed later.

PROJECTION PHASE (1960-2010)

Having once established the identity and volume of tonnage moving to or from the Great Lakes area during the base period (1955-1956) it is necessary to take a long-range look at the prospects of commerce as a whole and the selected commodities in particular. This projection for the period 1960-2010 is required in order to compare estimates of benefits and costs on an average annual basis for the assumed economic life of the project, that is usually set at 50 yr.

Thus, in regard to overseas commerce of the Great Lakes, it becomes necessary to make estimates in what is perhaps one of the most variable elements of the national economy—the prospective long-range level of general cargo in foreign trade. Stated in another way, it is a question of determining whether tonnage that contributes during the early years of the project to establish a favorable benefit-cost ratio can reasonably be expected to continue for the 50-yr assumed economic life of the project. Conversely, it may also be a question of determining whether tonnage that does not contribute to an adequate extent in the early years of the project, may build up to a sufficient level during the life of the project.

National, Great Lakes Area, and Port Traffic Potentials.—In making projections of future traffic a subjective approach is suggested, with analysis proceeding from the general to the particular. In all instances such projections will proceed first from analyses conducted at a national level, then to analyses at the "Great Lakes Area" level, and finally to analyses at local port levels.

Basically, the projections take into account such indicators as trends in gross national product (GNP), industrial production, and population to establish, insofar as data are possible, a reasonable indication of basic growth rates in the economies of the principal countries, or geographic areas accounting for the bulk of our foreign commerce. Using these national indicators as bench marks, projections can be made at a national level of trade for the eleven major commodity groups for which transportation differential data are being developed.

The projection estimates for these eleven commodities are based on analyses of the conditions of trade applicable to principal, or representative items in each of the eleven commodity groups. Inasmuch as there are some 400 commodities involved for exports and imports combined, only the most significant items of trade should be chosen for study.

For each of the commodities selected, a separate analysis is made as to historical trends in national production, long-range potential trends in the total of imports and exports, and historical relationships between the level of foreign trade and domestic production. Where available data permits, analysis also covers shifting levels of consumption and indicated changes between competing products, substitute products or both.

The resulting projections by commodity groups, represent estimates properly weighted to account for the relative importance of commodities in each category. These commodity group projections are used as a bench mark in making the estimates of the long run potential for foreign trade at a regional level.

It is to be emphasized that such estimates are not forecasts in the true sense of the word. Rather, after evaluating presently available data, and taking into account the indicated shifting composition of such trade, they should be considered as conservative levels of trade to be expected in the future.

Estimates Used.—In recognition that foreign trade is subject to substantial fluctuation from year to year, estimates of potential traffic will show high and low ranges between which the trade levels are likely to vary. Inasmuch as conservative estimates are used, the actual future level of trade may well be considerably higher.

Mention should be made also of commercial policy with regard to prospective levels of foreign trade because the question invariably arises, as to whether any forecast of trade can adequately take into account the possible imposition of mechanical devices to restrict trade for balance of payments, for protection or for other reasons. In this respect, for the purpose of deriving conservative minimum estimates of future trade levels, it can be assumed that there will be no increase in the historical relationships between exports and domestic production, and imports and levels of domestic consumption. Thus, forecasts will be based on growth prospects only and not provide for any increase in the share of the market commanded by foreign goods.

Moving from the national to the regional or Great Lakes area level, the national projections are evaluated on the basis of the relative importance of the producing-consuming sector of the regional economy, as indicated by regional-growth trends. This phase of the analysis also involves use of the regional economic data that the Corps collected for this purpose by means of interviews from many industries.

At a port level, the estimates of potential traffic represent an evaluation of information collected through staff research, that in turn is evaluated through first-hand discussion with industry officials actually engaged in foreign trade and through information developed in cooperation with local communities at public hearings.

Admittedly, in the light of the present status of the art of forecasting, projections are subject to a degree of error, but by taking what is considered a conservative position it is believed that a sound projection of the grade of the 1960-2010 period can be achieved.

TRANSPORTATION ANALYSIS PHASE

After determining the points of origin and destination of foreign trade in the United States, the next step is to establish the limits of the Great Lakes tributary area. This can be done by comparing the transportation expenditures for Great Lakes-Seaway routings and for coastal port routings and establishing a "differential" or "transportation advantage." The magnitude of this transportation advantage can be termed the "driving force" that would orient traffic toward the Great Lakes ports or toward the seaboard, assuming that factors relating to shipping service, terminal facilities and such, are equal at the alternative ports. The problem then is to develop a method of measuring this differential or relative transportation advantage.

The determination of transportation differentials requires at least 30,000 computations, or two for each of the 15,000 observations in the sample. Maximum simplification of these computations can be achieved through the use of algebraic formulas, punch cards and tabulating machines.

It is recognized that transportation differentials can be developed on the basis of both (a) freight rates, and (b) carrier costs. From the viewpoint of methodology, the treatment of rates or costs is very similar. The technique developed on the basis of carrier cost data will be explained herein.

The transportation cost for overseas export or import shipments is composed of four segments: (a) Ocean Transportation costs, (b) overland transportation costs, (c) tolls, and (d) cargo handling charges. In view of the long-term nature of the determinations being made, handling charges for the future period were assumed to be equalized for alternative routings. The level of tolls used are those announced by the United States and Canada. This leaves only the ocean and inland transportation costs to be determined.

Ocean Transportation Costs.—The ocean carrier's cost is based on the vessel's operating, administrative and capital expenses for a round voyage of a C-2 type ship on specified routes. This total route cast is translated into the cost per cubic foot of the vessel's utilized cargo space. By application of a factor representing the number of cubic feet of the vessel's cargo space occupied by a hundredweight of a specific commodity, the cost of transporting the hundredweight of that commodity can be determined. In other words, the ocean transportation cost per hundredweight (cwt) for a specific commodity to or from a designated foreign area is equal to the vessel cost per cubic foot of cargo space multiplied by a density stowage factor that represents the number of cubic feet per hundredweight of that commodity. Or to express it in a formula

$$\text{Ocean cost per cwt} = S = (\text{number of cubic feet per hundredweight})(\text{vessel cost per cubic foot}) \quad \ldots\ldots (1)$$

or

$$S = \frac{(\text{number of cubic feet per gross ton})}{(\text{number of hundredweight per gross ton})}(\text{vessel cost per cubic feet}) \quad \ldots (2a)$$

$$S = \frac{(\text{stowage factor})}{(2240 \text{ lb per } 100 \text{ lb})}(\text{vessel cost per cubic foot}) \quad \ldots\ldots (2b)$$

$$S = \frac{F}{22.4}\, k \quad \ldots\ldots (3a)$$

in which S is the ocean transportation cost per hundredweight for a specified commodity to a specified foreign area, F denotes the stowage factor for the specified commodity[2,3] (number of cubic feet of commodity per 2240 lb of commodity), and k represents the one-way vessel cost per cubic foot of cargo space for a specified routing. It is based on the total vessel cost for specified routing divided by number of cubic feet of space utilized for cargo. Toll charges against the vessel are included.

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[2] "Modern Ship Stowage," by Joseph Leeming, U. S. Dept. of Commerce, 1942.

[3] "The Stowage Red Book," compiled and edited by Harry R. Hanlin, Walter W. Weller and Charles J. Fagg, Traffic Publishing Co., Inc., New York, 1944.

To establish formulas for the ocean transportation cost via a coastal routing compared to a Seaway routing, algebraic symbols are provided for designated items. With symbols associated with the Great Lakes-Seaway routings distinguished from ocean transport via a seaboard port by the prime accent mark used in the former, the ocean transportation costs are, via a coastal port

$$S = \frac{F}{22.4} k \qquad (3b)$$

and via the Great Lakes-Seaway

$$S' = \frac{F}{22.4} k' \qquad (3c)$$

To establish the cost per cubic foot of vessel cargo space (k), the following formulas are developed:

$$\text{Cost per cubic foot} = k = \frac{V}{n} = \frac{\text{vessel cost for designated route}}{\text{number of cubic feet of cargo space}} \qquad (4)$$

in which

$$V = d_s v_s + d_p v_p + Z \qquad (5)$$

and in which d_s is the number of days at sea, v_s denotes the vessel cost per day at sea, d_p refers to the number of days in port, v_p is the vessel cost per day in port, Z refers to the other costs and charges related to the vessel, such as tolls on the vessel, pilotage, wharfage, and so on, and n is the number of cubic feet of vessel cargo space utilized on the specified route. Therefore,

$$k = \frac{V}{n} = \frac{d_s v_s + d_p v_p + Z}{n} \qquad (6)$$

If k is vessel cost per cubic foot for shipment via a coastal port, and k' denotes the vessel cost per cubic foot for shipment via the Great Lakes-Seaway then

$$k = \frac{V}{n} = \frac{d_s v_s + d_p v_p + Z}{n} \qquad (7a)$$

$$k' = \frac{V'}{n} = \frac{d'_s v_s + d'_p v_p + Z'}{n} \qquad (7b)$$

and

$$S = \frac{F}{22.4\, n} \left(d_s v_s + d'_p v_p + Z'\right) \qquad (8a)$$

$$S' = \frac{F}{22.4\, n} \left(d'_s v_s + d'_p v_p + Z'\right) \qquad (8b)$$

Overland Transportation Cost.—For the purpose of the determination of transportation cost differentials, the overland transportation costs can be represented by railroad costs. In the USBC origin-destination study, it was found that the railroads transported the greatest proportion of the shipments. The railroad cost formulas used are those developed and published by the ICC. They represent costs based on 1956 operations with adjustments to reflect wage and price levels as of January 1, 1958.[4]

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[4] "Rail Carload Cost Scales by Territories as of January 1, 1958," Statement No. 2-58, Interstate Commerce Comm., Washington, D. C.; prepared by The Cost Finding Sect., Bur. of Accounts, Cost Finding and Valuation, March, 1958.

The rail costs are composed of terminal costs plus line-haul costs and can be expressed as

$$\text{Rail cost} = (\text{Terminal cost}) + (\text{Line-haul cost}) \quad \ldots\ldots(9a)$$

or

$$R = \left(\frac{T}{c} + t\right) + X\left(\frac{H}{c} + h\right) \quad \ldots\ldots\ldots\ldots\ldots(9b)$$

in which R is the rail costs per hundred weight of commodity, T denotes the per carload part of the terminal cost (varies with type of car), c refers to the number of hundredweights per carload (varies with commodity), t is the per hundredweight part of the terminal cost, X refers to the distance of haul, H denotes the per car-mile part of the line-haul cost (varies with type of car), and h is the per hundredweight-mile part of the line-haul cost, then rail cost to coastal port is determined from Eq. 9(b), and rail cost to the Great Lakes port is

$$R' = \left(\frac{T}{c} + t\right) + X'\left(\frac{H}{c} + h\right) \quad \ldots\ldots\ldots\ldots\ldots(9c)$$

in which X is the distance from inland point to seaboard port, and X' denotes the distance from inland point to Great Lakes port.

The rail costs, whether measured either by out-of-pocket costs (long-run variable costs) or by fully distributed costs (long-run fixed and variable costs) can be computed by Eq. 9(a). To establish comparability between the factors included in the rail costs and the ocean costs, adjustments can be made in the cost data. As will be noted, several cost elements for each type of carrier drop from consideration in the final determination of the transportation differential.

As part of the method of utilizing the most economical routing of a shipment, barge costs and truck costs were considered always with rail costs. The determination of carrier costs for the barge shipments were based on a method similar to that used in the ocean-cost determination. Truck costs are examined to determine if such costs yield a more economical routing for certain shipments.

As part of the problem of handling the great mass of data, the method used to obtain the inland distance of the 30,000 shipments to the actual port of exit or entry is designed for machine computation. Each geographical point in the study (a port or an inland city) is located on a mile grid. The straight line distance between points is determined by means of coordinates on the mileage grid. Then, straight line distances are expanded to short line rail distances that, in turn, are adjusted for circuitry to obtain an approximation of actual distance.

Transportation Cost Differentials.—In determining the transportation cost differentials, the question of cargo handling costs becomes a problem in determining the difference in cargo handling costs for any two routes being compared. Because the foreign terminus of a particular alternate route is the same, and because the costs for handling the same cargo at the Great Lakes port and United States coastal port, should be, over the long run, essentially the same, such cargo handling costs cancel out and are, therefore, omitted as a factor in determination of the transportation-cost differential. In addition to cargo handling costs, certain tolls are assessed against the ship's cargo when the routing is via the Seaway, the Suez Canal, or the Panama Canal.

To express the toll charges in algebraic terms, let L be the tolls for routings via United States coastal ports, and L' signify the tolls for routings via Great Lakes ports.

Summary.—The total transportation cost for each routing is the sum of the overland, ocean and tolls segments. If Y is the total transportation cost via a coastal port, and Y' denotes the total transportation cost via the Great Lakes-St. Lawrence, then

$$Y = S + R + L \text{ (ship cost + rail cost + tolls)} \ldots\ldots (10a)$$

and

$$Y' = S' + R' + L' \text{ (ship cost + rail cost + tolls)} \ldots\ldots (10b)$$

The differential, D is

$$D = Y - Y' \ldots\ldots\ldots\ldots\ldots\ldots (11a)$$

or

$$D = (S + R + L) - (S' + R' + L') \ldots\ldots\ldots\ldots (11b)$$

If D is positive (+) the cost advantage is via the Great Lakes, whereas if it is negative (-) the cost advantage is via the seaboard.

If the expressions for the components of the total cost are substituted in Eq. 11(b) and the expression is simplified by assuming that the number of days in port is the same by either routing $(d_p = d'_p)$, the differential can be reduced to the expression

$$D = \frac{F}{22.4\,n}\left[v_s\left(d_s - d'_s\right) + \left(Z - Z'\right)\right] + \left(X - X'\right)\left(\frac{H}{c} + h\right) + \left(L - L'\right) \ldots (12)$$

These formulas establish the values of the differential D for each of the 15,000 transactions in the USBC origin-destination study. Applied to location of traffic these determinations can be used to estimate the volume of the base period (1955-1956) traffic with a transportation cost advantage via the Great Lakes-Seaway. These data can also be utilized to construct "differential contour" patterns (lines of the same advantage) that can be applied to the distribution of the projected foreign trade origins and destinations. The differential contour pattern depicts the specified "driving force" (such as 25 cents per cwt, 50 cents per cwt, or other amount) for a particular shipment to indicate the transportation advantage for that shipment via the Seaway. It is, thus, possible to show the volume of foreign trade within specific categories of transportation advantage for a routing via the Great Lakes-St. Lawrence Seaway and the connecting channels. Conversely, advantages via seaboard ports will also be apparent.

The ocean distance, travel time, and carrier cost between Great Lakes ports and overseas ports are somewhat greater than for alternate routes through most seaboard ports. These differences are often reflected in existing ocean freight rates. When considered in conjunction with the transportation differentials previously examined, it becomes more apparent that the advantages of the Great Lakes-St. Lawrence route may be largely the difference in the overland haul. In general freight rates and carrier costs show that the greatest transportation advantage via the Seaway is for shipments originating or terminating in or near Great Lakes ports. The next advantage zone includes those cities at a substantially greater distance from a coastal port than from a Great

Lakes port. The least advantage is shown where points of origin or destination are about the same distance from coastal port and Great Lakes port.

POTENTIAL TRAFFIC PHASES (1960-2010)

The preceding sections have shown how methodology is developed to obtain basic data regarding (1) the magnitude of foreign trade in the base year period (1955-1956) and its projection into the 1960-2010 period, (2) the location of the origin and terminus of the exports and imports comprising this foreign overseas trade in the United States, and (3) the cost of transporting this overseas cargo by land and by sea between points of origin and destination. The foregoing factors are brought together in the potential traffic phase so that the volume of overseas traffic generated in the Great Lakes Tributary area is analyzed in terms of the probable volume that may actually transit the Seaway. This estimate of total potential Seaway traffic moving via United States Great Lakes ports is based largely on the findings of the analysis of the most economical routings either through coastal ports or by Great Lakes ports. The traffic estimate is refined by considering more subjective factors as the "Seaway season" and institutional factors affecting commodity movements.

Finally, the volume of potential Seaway traffic must be further delineated by estimating the volume of traffic at individual harbors on the Great Lakes. The methodology applied in allocating the total potential Seaway traffic to individual Great Lakes harbors represents, in miniature, the determination of the Seaway traffic volume for the Great Lakes area as a whole. That is, the tributary area for each port is delimited in terms of the traffic generated within the port's general area of influence, the volume that can most economically move through the particular port is determined, and, finally, the port's potential traffic is estimated. Again, the final estimate of potential Seaway traffic is refined in terms of more subjective considerations that include the local condition at each port in terms of facilities and institutional forces that affect commodity movements.

In many cases it is impossible to determine through which port a commodity will move, because it is probable that any of two and perhaps three ports might be used at different times. Thus, port traffic estimates must also contain an element of judgment beyond the mechanical computations by which a particular estimate is derived.

SUMMARY AND CONCLUSIONS

In summary then, the objective has been to obtain information as comprehensive in scope and as factual as possible for use in estimating traffic potentials to evaluate requirements for harbor improvements.

As has been seen, the methodology moves through several successive stages of analysis at each of which the data are subjected to a greater degree of refinement.

The initial analysis is directed primarily at the national level and efforts concentrated on identifying the important volume items in total United States overseas trade. Analysis is then narrowed to the Great Lakes tributary area and attention focused on the share of the total national overseas traffic generated by this region.

Consideration is next given to the probable future level of trade in commodities significant to the Great Lakes region. Estimates of such traffic are then tested for the most economical routing. From analyses of data collected in field interviews and from staff research, an evaluation is then made of other factors likely to influence the use of the Great Lakes. From a synthesis of all these data an informed estimate as to the total potential tonnage that may move via the Great Lakes, the connecting channels and the St. Lawrence system becomes possible. Following the determination of the Great Lakes-overseas traffic potential, an estimate can be made of the traffic expected to move through the individual lake ports. This estimate is based upon consideration of a variety of factors, such as transportation-cost differentials, the magnitude of the economic hinterland of individual ports, and shippers' opinions as to possible future routings of the more important items of commerce.

The sources of information used in an analysis of this nature are varied and extensive. Not only is a wealth of Government statistics available as basic background data, but many Government agencies have been most cooperative in supplying unpublished information. The Department of the Interior, for example, furnished information on the present and prospective flow of minerals in the Great Lakes area; and the Department of Agriculture supplied valuable information on many agricultural products.

Of particular interest, because it proved to be an important key to basic problems of the Great Lakes-overseas traffic survey, has been the opportunity to analyze on a nation-wide basis the origins and destinations in the United States of foreign trade cargo. Through this origin-destination study it has been possible to develop much transportation information heretofore not available. However, as previously stated any formulas or methods proposed in connection therewith, but not yet accepted by the United States Army Corps of Engineers, reflect only the views of the author and not necessarily those of the Corps.

The interest of the Corps of Engineers in traffic studies such as the present one is to obtain a comprehensive view of traffic potentials for the purpose of evaluating the "economic justification" of proposed harbor projects.

At this time, note should be taken of some of the problems that have been encountered in the study. For example, the lack of a continuing "foreign trade census" prevents development of historical perspectives as to the regional importance and trends of international trade and commerce. The rather extensive data available as to the pattern of freight movements by the railroads is not complemented by comparable data regarding freight movements by truck.

In setting forth a methodology for determining overseas traffic potentials, this analysis is offered with a view to inducing others to explore further the possibilities presented here. Such future possibilities may well include a continuing census of the inland origins and destinations of overseas foreign trade generated in the United States.

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PORT STRUCTURES IN LONG BEACH HARBOR

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SYNOPSIS

The comparatively rapid growth of the port of Long Beach (Long Beach, Calif.) is guided by a long-range master plan. The purpose of this plan is to provide an efficient and self-sustaining port that will meet the present and future needs of the surrounding area.

The preparation of the master plan has required the development of certain basic planning and design criteria for port structures.

BRIEF HISTORY OF PORT

Development of the port of Long Beach (Long Beach, Calif.) began in 1905. In that year private Long Beach interests acquired 800 acres of marshland in the Inner Harbor area and initiated a program of channel dredging and land reclamation. Efforts to establish a municipal port began a short time later and culminated in the opening of pier 1 in 1911.

An event of even greater significance to the port future also occurred in 1911. This was the tideland grant enactment by the state legislature that conveyed to the city of Long Beach all of the state's rights in the tidelands located within the city boundaries. The grant stipulated that all revenue produced from the tidelands be used for the development of navigation, commerce, and fisheries.

During the next 15 yr, up to 1926, the port's commercial growth was steady, although relatively slow. This period saw the accomplishment of several fun-

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damental steps in the development of a major harbor. These included the creation of the board of Harbor Commissioners in 1917, the dredging of the Inner Harbor and entrance channel to accommodate deep draft vessels, and the correction of a serious silting problem through construction of the Los Angeles River Flood Control Channel. These efforts bore fruit during the ensuing 10 yr in the establishment of more than $30,000,000 worth of industrial plants and private marine terminals in the Inner Harbor. Commerce over municipal wharves also increased more rapidly and it became necessary to start the development of an Outer Harbor.

In 1936, oil was discovered in the harbor area, and the first Harbor Department well was brought in 2 yr later. It soon became evident that the city of Long Beach would receive substantial revenue that, under the terms of the state grant of 1911, must be used in the development of the port. It was further realized that to properly guide and coordinate such development, a long range comprehensive plan was essential. The Board of Harbor Commissions contracted with George F. Nicholson, M. ASCE, and James F. Collins, consulting engineers, for the preparation of such a report, that was completed in 1940.

The Nicholson-Collins report proposed the development of an East Basin between the entrance channel and the Los Angeles River, and a West Basin in the area now occupied by the Long Beach Naval Shipyard. Twenty full-size berths were to be provided in the East Basin and more than fifty berths in the West Basin. The report recommended extensive modern terminal facilities for handling waterborne cargo. Attention was also given to improvement of the street and highway system to give better vehicular access and to avoid congestion within the port.

To a large extent the recommendations of the Nicholson-Collins report were followed in the subsequent development of the East Basin. This development was greatly accelerated by World War II, with its need for increased oil production and additional shipping facilities. The wartime and immediate postwar construction was largely concentrated on pier A, with the development of piers B, C, and D following in more recent years. An important factor in the planning of these piers was their dual function as oil drilling sites and cargo piers.

The Navy's acquisition of the West Basin area made it necessary to depart from the Nicholson-Collins plan and find other areas for future expansion. At the present time, only one Inner Harbor Post-owned area having protected deep water frontage remains undeveloped. This is the property fronting on Channel 2 in the Inner Harbor, acquired in December 1957, from Pacific Dock and Terminal Company.

EXISTING FACILITIES

The fairway to the port of Long Beach starts at an 1,800-ft-wide opening in the federal breakwater, 3 miles offshore. This 750-ft-wide channel has recently been dredged to a minimum depth of 52 ft in order to allow supertankers to enter the port, see Fig. 1.

Generally, the minimum depth of water in the channel, basins, and slips in the port is maintained at 40 ft at mean lower low water. The minimum depth of water at pierhead line is maintained at 35 ft. Consequently, all general cargo ships can enter the port at all phases of the tide.

The width of slips in the Middle Harbor, that was constructed over 15 yr ago, was set at 400 ft. This width enables a cargo ship to navigate the slip with ships tied up at the adjacent side berths. However, in the Outer Harbor, that is now under construction, 500-ft-wide slips are being used. These slips are four berths long. Some of the reasons for expanding to 500-ft-wide slips were that this allowed room for a full berth at the ends and created safer maneuvering room for ships. In addition, the area could be economically obtained.

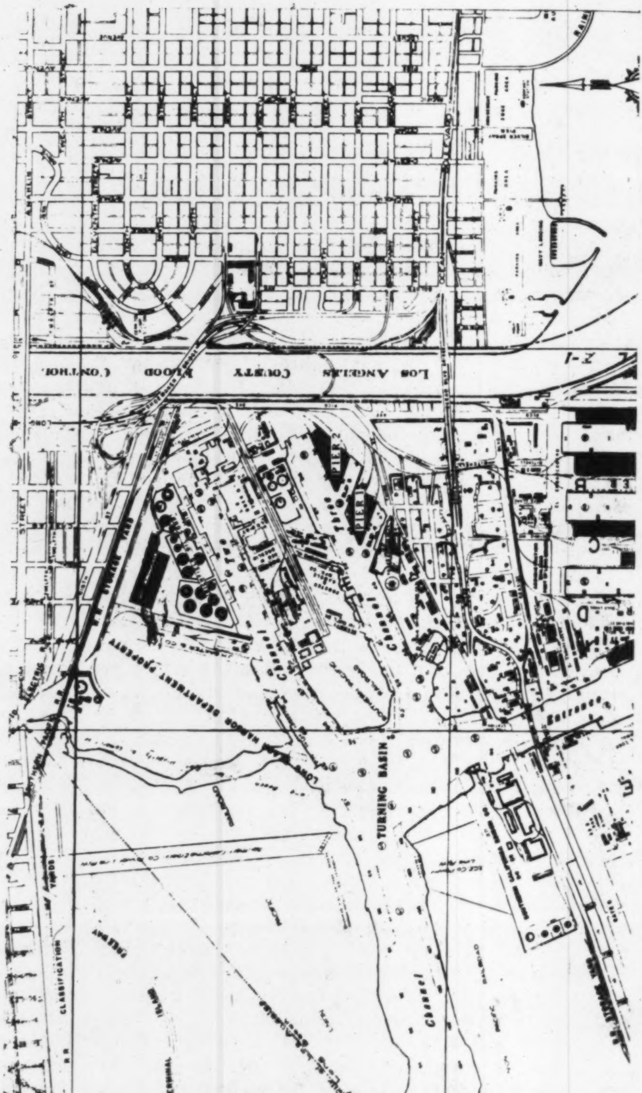
All of the piers in the port of Long Beach south of Seaside Boulevard are on filled land. This fill was obtained mainly by dredging in the ocean bottom. However, large quantities of fill have also been obtained from borrow areas outside the harbor district that were primarily being developed as "cut and fill" trash dumping pits.

The peris in the East Basin were completed in the following order: pier A, pier D, pier C, and pier B. Pier A, berths 1 through 10 can, for purposes of comparison, be divided into two parts. The physical features of berths 1 through 5 consist of 35-ft-wide wharves with tracks, 120-ft-wide clear span transit sheds, loading platforms, low level tracks, and minimum back areas for parking and storage. The physical features of berths 6 through 10 consist of 50-ft-wide wharves with tracks, 200-ft-wide clear span transit sheds, loading platforms, low-level tracks, and large back areas with warehouses. Records of the annual tons of cargo handled by these ten general cargo berths, indicate that berths 6 through 10 handle approximately twice as much general cargo as berths 1 through 5. The layout of the new Outer Harbor, of which Fig. 2, is an example, resulted from a careful study of the Middle Harbor. Conclusions of this study were, that for maximum operating efficiency, a berth should have a 50-ft-wide wharf with tracks, at least a 160-ft-wide clear span transit shed (80,000 sq ft per berth), a 16-ft-wide loading platform, low level tracks, a back area storage space of approximately 50,000 sq ft with warehousing as needed, and adequate car parking facilities.

Several types of wharves have been used in the port development. These are shown on Figs. 3, 4, and 5. Piers B and C were constructed using the wharf section shown on Fig. 3. This style of wharf embodies the use of a steel sheet pile bulkhead with a tieback system, precast concrete piles, and reinforced concrete deck covered with $3\frac{1}{2}$ ft of fill. A cathodic protection system is used on the steel sheet pile bulkhead. This system has proved satisfactory for initial construction in quiet waters.

For construction in areas in which the initial construction was subject to more open water conditions, the steel cellular bulkhead shown in Fig. 4 has been used. Each cell when filled with sand becomes a stable element in itself. Concrete caps are added later to finish the section for berthing of ships. This system has been satisfactorily used for the construction of nine berths. However, one of the chief problems of this type of bulkheading is the lining up of the cells during construction. Experience shows that the alinement may be off as much as 6 ft. This results in a higher cost for the concrete cap.

Another type of bulkheading that has been used successfully for ten berths is a cyclopean concrete quay, as shown in Fig. 5. These berths were all constructed in quiet waters. Although this bulkhead is high in first costs, the life of the structure might conceivably be over 100 hr. However, the port usually considers a 50-yr obsolescence for structures. An advantage of a concrete quay is that it requires little steel and, therefore, may be constructed during periods of national emergency. Also, because of their mass, they have an inherent strength against unusual forces.


Los Angeles County Flood Control
Z-1
Pier Two
Pier Three
Channel
Channel
Channel
Turning Basin
Entrance
Long Beach Harbor Department Property
Terminal Island
Classification Yards
R.R. Storage Yard
R.R. Storage Yard
Direct Truck Routes to Los Angeles Industrial Districts
Boulevard
Anaheim Street

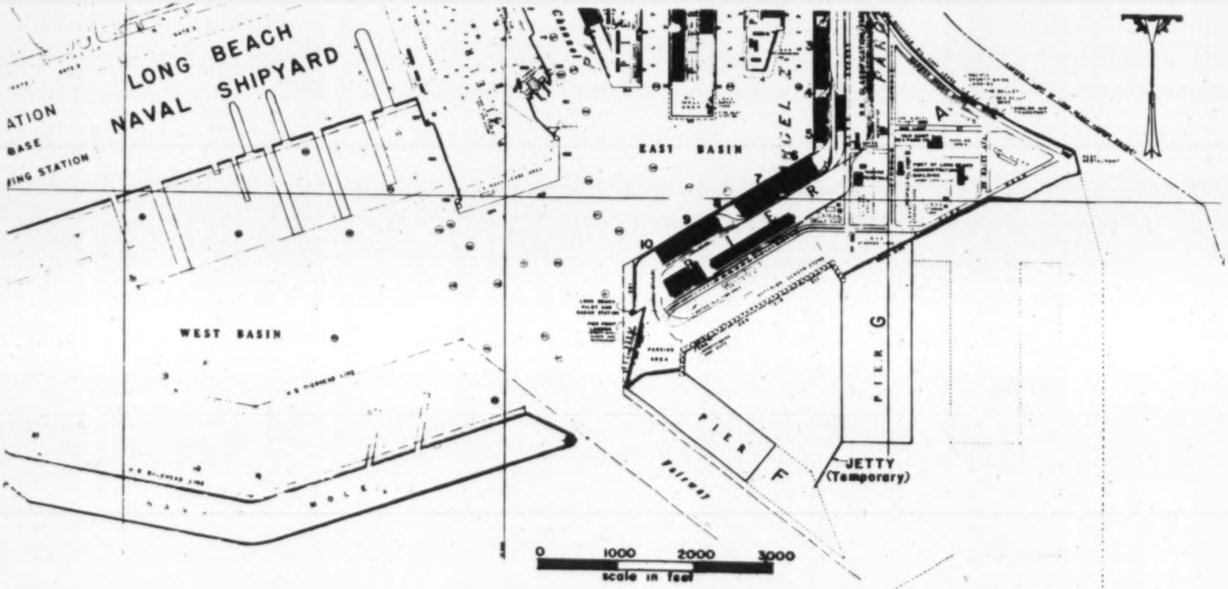
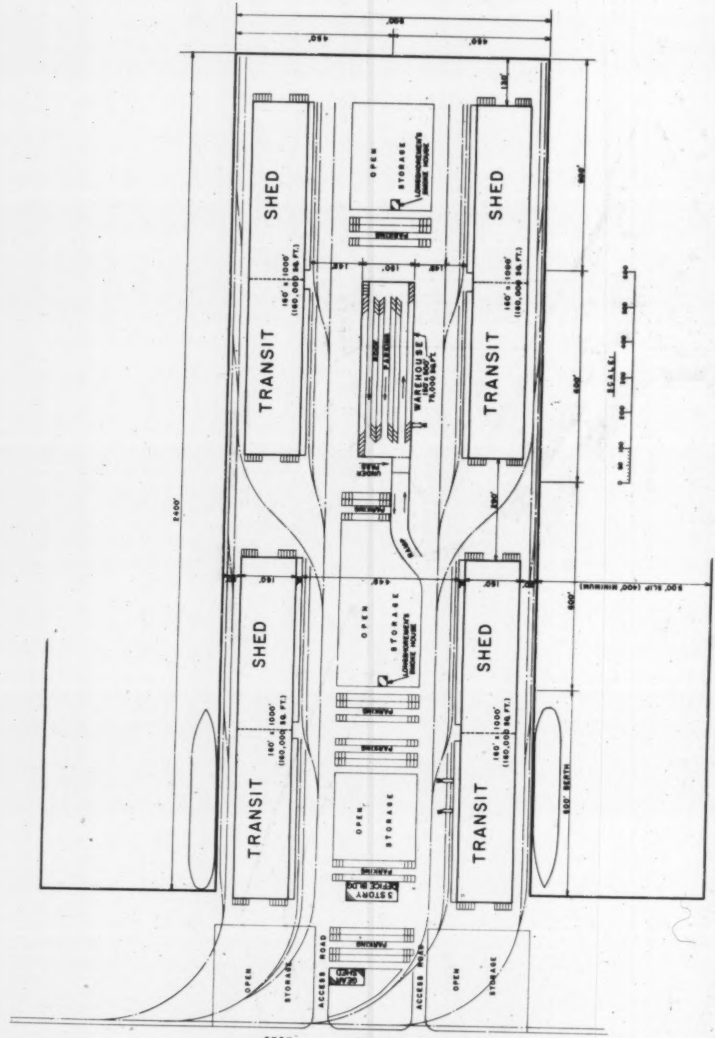
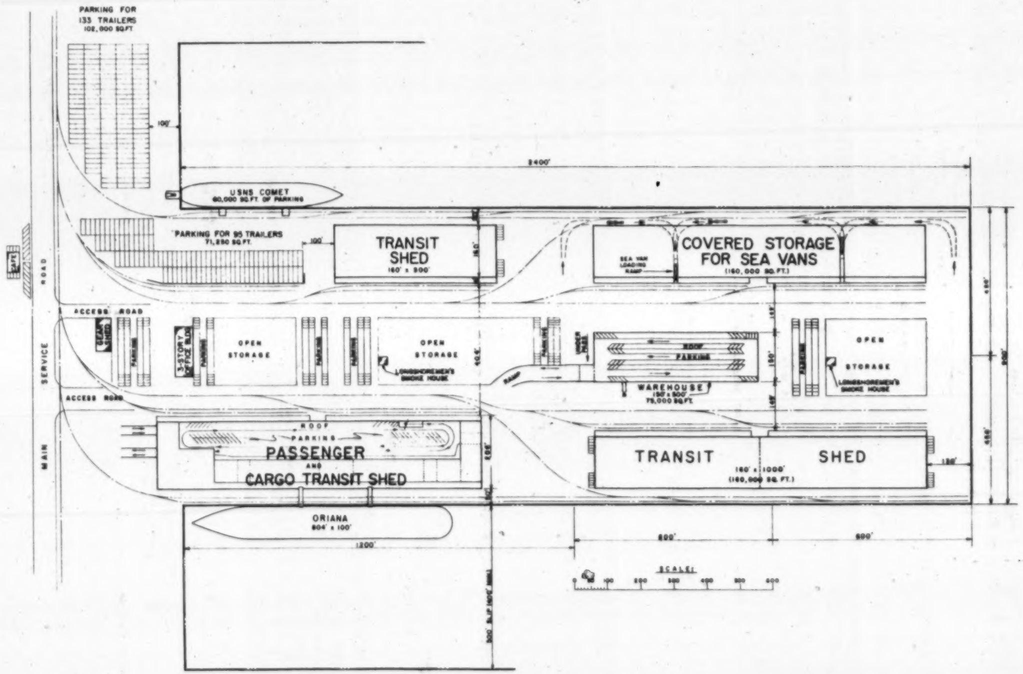

LONG BEACH
NAVAL SHIPYARD
ATION
BASE
ING STATION
Channel
EAST BASIN
WEST BASIN
PIER A
PIER E
PIER F
PIER G
JETTY
(Temporary)
NAVY MOLE
Fairway
U.S. PIERHEAD LINE
U.S. BULKHEAD LINE
PARKING AREA
3
4
5
6
7
8
9
10
0
1000
2000
3000
scale in feet

FIG. 1.—PORT OF LONG BEACH, CALIF.


TRANSIT SHED
160' x 1000' (160,000 SQ. FT.)
WAREHOUSE
150' x 500'
75,000 SQ.FT.
OPEN STORAGE
LONGSHOREMEN'S SMOKE HOUSE
PARKING
UNDER PASS
RAMP
3 STORY OFFICE BLDG.
GEAR SHED
ACCESS ROAD
MAIN SERVICE ROAD
500' SLIP (400' MINIMUM)
600' BERTH
2400'
900'
450'
150'
130'
600'
SCALE:

(a)


PARKING FOR 133 TRAILERS 102,000 SQ.FT.
100'
2400'
USNS COMET 60,000 SQ.FT. OF PARKING
PARKING FOR 95 TRAILERS 71,250 SQ.FT.
100'
TRANSIT SHED 160' x 900'
150'
COVERED STORAGE FOR SEA VANS (160,000 SQ.FT.)
SEA VAN LOADING RAMP
CAFE
ROAD
SERVICE
MAIN
ACCESS ROAD
ACCESS ROAD
GEAR SHED
PARKING
3-STORY OFFICE BLDG
OPEN STORAGE
OPEN STORAGE
LONGSHOREMEN'S SMOKE HOUSE
404'
RAMP
UNDER PASS
ROOF PARKING
WAREHOUSE 150' x 500' 75,000 SQ.FT.
145'
150'
145'
OPEN STORAGE
LONGSHOREMEN'S SMOKE HOUSE
ROOF PARKING
PASSENGER AND CARGO TRANSIT SHED
305'
TRANSIT SHED 160' x 1000' (160,000 SQ.FT.)
120'
450'
900'
450'
ORIANA 804' x 100'
1200'
300' SLIP (400' MIN.)
600'
600'
SCALE:
0 50 100 200 300 400 500 600

(*b*)

FIG. 2.—TYPICAL PIER LAYOUTS, OUTER HARBOR

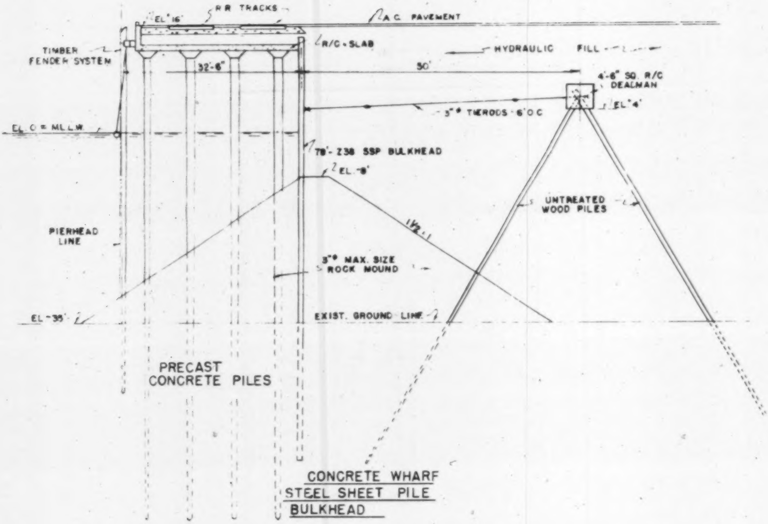

EL 16'
RR TRACKS
A.C. PAVEMENT
TIMBER FENDER SYSTEM
R/C SLAB
HYDRAULIC FILL
32'-6"
50'
4'-6" SQ. R/C DEADMAN
EL 4'
3" Ø TIERODS 6' O.C.
EL 0 = MLLW
78'-Z38 SSP BULKHEAD
EL -8'
UNTREATED WOOD PILES
PIERHEAD LINE
1½:1
3" Ø MAX. SIZE ROCK MOUND
EL -35'
EXIST. GROUND LINE
PRECAST CONCRETE PILES
CONCRETE WHARF
STEEL SHEET PILE
BULKHEAD

FIG. 3.—WHARF SECTION USING STEEL SHEET PILE BULKHEAD

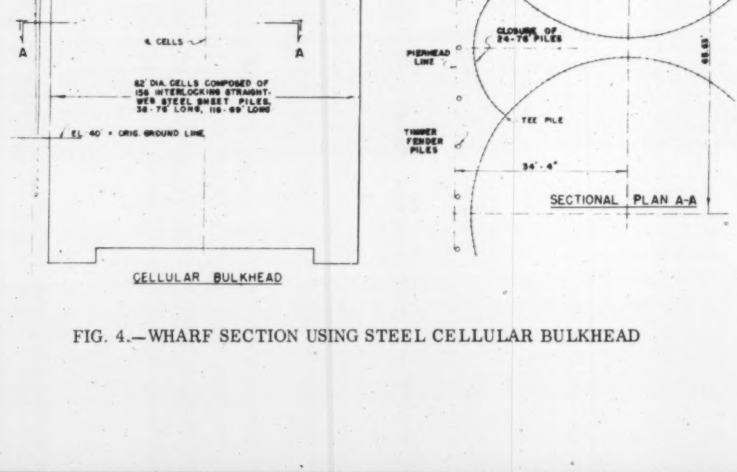

PIERHEAD LINE
RR TRACKS
EL 16'
A.C. PAVEMENT
R/C CAP
Rock Ballast
3'
EL 8.5'
3'
EL 0 = MLLW
HYDRAULIC FILL
A
℄ CELLS
A
62' DIA. CELLS COMPOSED OF 156 INTERLOCKING STRAIGHT-WEB STEEL SHEET PILES, 38-76' LONG, 118-69' LONG
EL 40' = ORIG. GROUND LINE
CELLULAR BULKHEAD
℄ CELLS
TEE PILE
CLOSURE OF 24-76' PILES
PIERHEAD LINE
48.63'
TEE PILE
TIMBER FENDER PILES
34'-4"
SECTIONAL PLAN A-A

FIG. 4.—WHARF SECTION USING STEEL CELLULAR BULKHEAD

The typical transit shed in the port is a clear span, steel rigid frame, concrete wall building that is fully sprinklered and has been designed to meet the requirements of the National Board of Fire Underwriters. These sheds have 18-ft-wide-by-16-ft-high steel rolling doors on the sides, approximately 30 ft on centers. The sheds vary in size from approximately 60,000 sq ft per berth at berths 1 and 2, to 108,000 sq ft per berth at berths 6 and 7. A typical example is shown in Fig. 6.

As of 1960, the port has six warehouses and has concluded that in order to meet the needs of shippers, back-area warehousing is a necessity. The latest warehouses have areas of over 100,000 sq ft and are divided into three rooms with 1-ft-thick firewalls as shown in Fig. 7. These warehouses are constructed with tapered steel girders, concrete tilt-up walls, and are fully sprinklered. The warehouses cost less than $3.50 per sq ft.

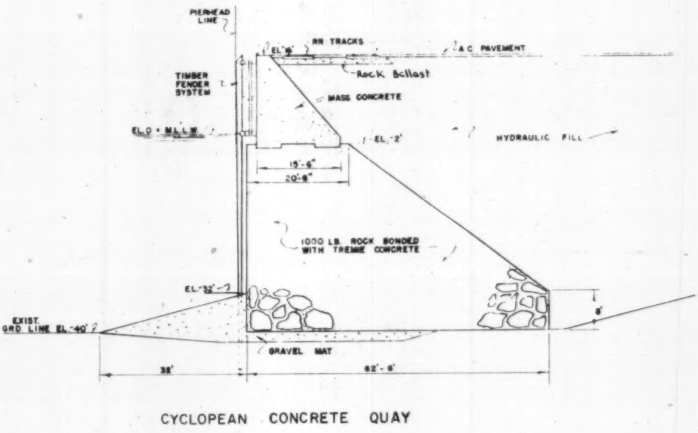

PIERHEAD LINE
RR TRACKS
EL.16'
A.C. PAVEMENT
Rock Ballast
TIMBER FENDER SYSTEM
MASS CONCRETE
EL.0 = M.L.L.W.
EL.-2'
HYDRAULIC FILL
15'-6"
20'-6"
1000 LB. ROCK BONDED WITH TREMIE CONCRETE
EL.-32'
8'
EXIST. GRD. LINE EL.-40'
GRAVEL MAT
32'
62'-6'
CYCLOPEAN CONCRETE QUAY

FIG. 5.—WHARF SECTION USING CYCLOPEAN CONCRETE QUAY

The first phase of the Outer Harbor is now under construction. This includes piers F and G and will furnish the harbor with ten new berths. In planning these piers an analysis of eight different types of wharves and bulkheads was made to determine the most economical design. All the previously used wharf types, as well as several new ones, were considered in the analysis. The cross section shown in Fig. 8 proved to be the most economical. This type is well suited to construction in open water in that the rock dikes and filling can be constructed first. The concrete wharf may then be constructed in quiet waters.

Land transportation to a port is as necessary as water transportation. Little value will result from fast ship-turnaround time if trucking and rail transportation is so bottled-up by inadequate arteries that they cannot keep up with the flow of cargo. For this reason, the port of Long Beach built three new high-

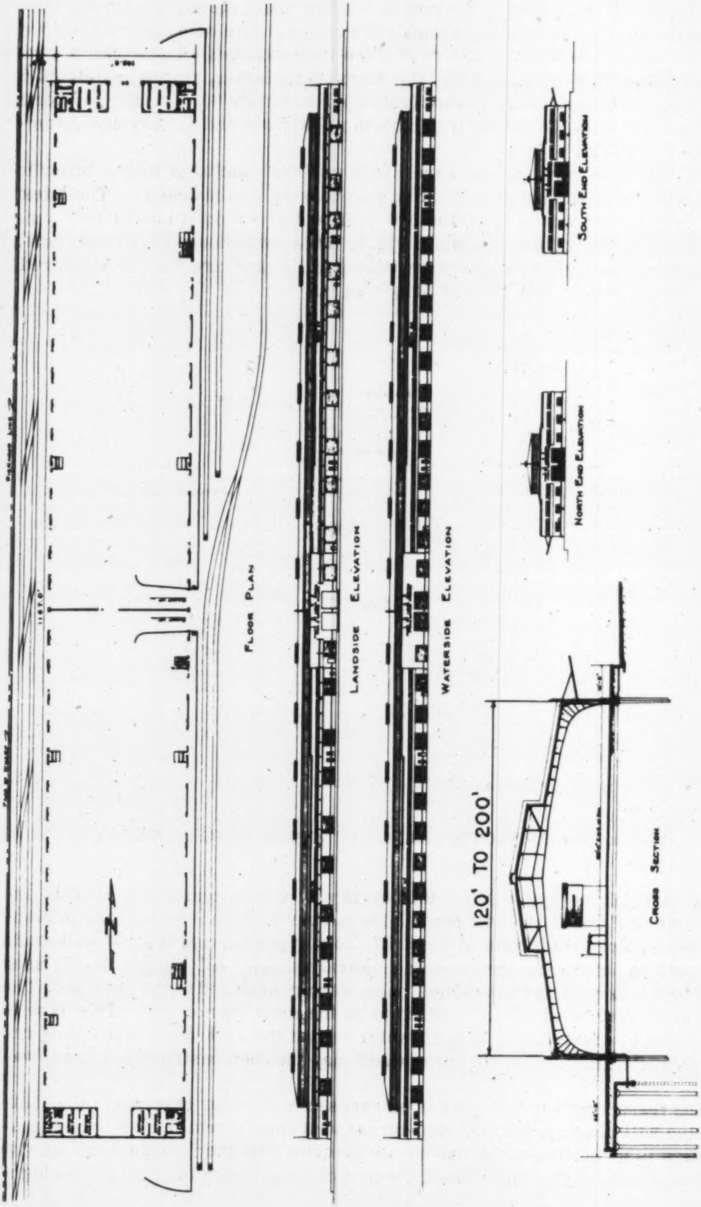

Floor Plan
Landside Elevation
Waterside Elevation
South End Elevation
North End Elevation
Cross Section
120' TO 200'

FIG. 6.—TYPICAL TRANSIT SHED

way bridges, is extending the local Freeway system into the port, and is revamping the railroad approaches to the port.

FUTURE DEVELOPMENT TRENDS FOR SHIPSIDE FACILITIES

Long range planning for the port of Long Beach is directed toward three main objectives:

1. To accommodate future increases in volume and changes in character of port traffic;
2. to provide for expansion of oil field operations; and
3. to make provision for necessary major traffic arteries into and through the harbor district.

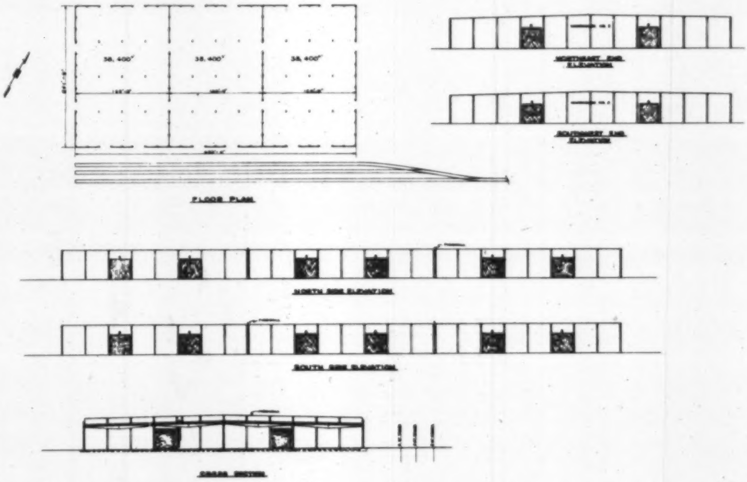


FIG. 7.—TYPICAL PORT WAREHOUSES

A study of future development trends for shipside facilities should not only look into new equipment and handling procedures, but should also review future growth possibilities of the port, capacities of existing facilities, and economics of operation.

Fig. 9 compares the growth rates of population and business activity in Southern California with the increase in general cargo handled by the port of Long Beach. It can be seen that since 1930, business activity and port commerce have each experienced a sixfold increase, whereas population has been multiplied by two and one-half.

From the evidence of past records, it would be over-conservative to predict future port needs on the basis of population growth alone. This can be seen from the fact that in Los Angeles and Orange Counties, since 1940, industrial

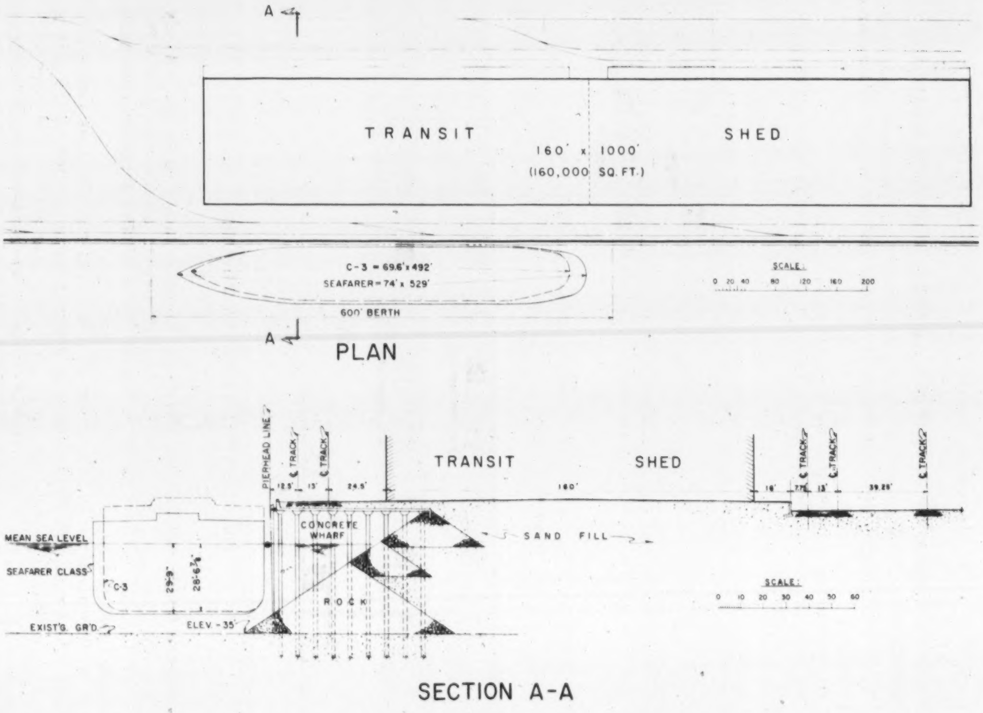

A
TRANSIT SHED
160' x 1000'
(160,000 SQ. FT.)
C-3 = 69.6' x 492'
SEAFARER = 74' x 529'
600' BERTH
SCALE:
0 20 40 80 120 160 200
A
PLAN
PIERHEAD LINE
℄ TRACK
℄ TRACK
12.5'
13'
24.5'
TRANSIT SHED
160'
16'
7.75'
13'
39.25'
℄ TRACK
℄ TRACK
℄ TRACK
CONCRETE WHARF
SAND FILL
MEAN SEA LEVEL
SEAFARER CLASS
C-3
29'-9"
28'-6½"
ROCK
EXIST'G. GR'D.
ELEV. -35'
SCALE:
0 10 20 30 40 50 60
SECTION A-A

FIG. 8.—MOST ECONOMICAL TYPE OF BERTH

activity, as measured by employment in manufacturing, has increased by 270%. This compares to a population increase of 78% in the same period and area.

There are many factors that point to the continuation of this growth in the future. Among these factors are the continuing westward movement of population and industry throughout the country and the wealth of natural resources in the western United States remaining to be developed. Another important consideration is the fact that much space remains in the Los Angeles metropolitan area for further population and industrial growth, in contrast to the very limited room for expansion in many other port areas.

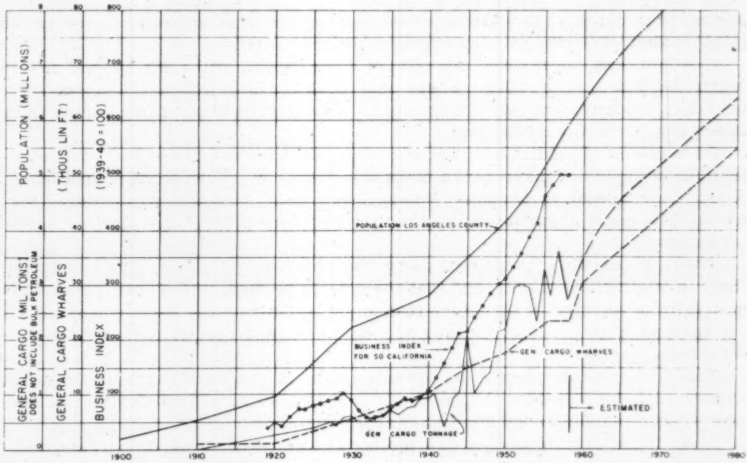


FIG. 9.—GROWTH OF LONG BEACH HARBOR

On the other hand, because the harbor's future growth will be influenced by many unpredictable factors such as the stability of international relationships technical improvements in transportation and cargo handling, labor practices, and world markets, a reasonable degree of conservatism is desirable.

In the absence of other supporting evidence in the form of future port commerce is based primarily on the predicted population growth, with conservative adjustment to allow for the past growth relationships of industry and port commerce. This projection indicates a required capacity for general cargo at the port of Long Beach of 6,000,000 tons per yr by 1980. This estimate appears conservative, inasmuch as it represents an average future increase of only 3.5% per year, whereas the past growth rate has averaged nearly 8% per yr over the past 30 yr (since 1930).

The second primary factor in estimating future port needs is that of berth efficiency, or the future tonnage capacity per berth. During 1957, ten general cargo berths handled an average of 94,000 tons per berth.

The cargo loading rate for ships at municipal berths averaged 940 tons per ship per day during 1954 and 1956, as compared to the estimated practical maximum loading rate of the Department of the Army of 540 tons per day.

In order to handle the projected 1980 general cargo volume of 6,000,000 tons per yr, if a rounded-off estimate of 100,000 tons per berth per yr is used, it would require sixty general cargo berths by 1980.

In addition to its general cargo capacity, the future harbor must also make adequate provision for other types of cargo and for a wide variety of port activities. Included among many others are additional bulk terminals with storage elevators, additional bulk oil terminals, fuel docks, passenger terminals, seaplane facilities, explosives terminals, berths for marine contractors, and repair facilities. Although future requirements for these facilities cannot be projected in detail, it is estimated that they will require at least one-fourth the total number of berths, or say twenty additional berths.

It would thus appear that the port of Long Beach will need approximately eighty berths total by 1980. There is a possible development of about forty berths in the Inner and Middle Harbors. This leaves about forty berths to be developed in the Outer Harbor.

As a means of guiding current planning effort toward these long-range objectives, a series of master plan maps was prepared. These show, respectively, 5-yr, 10-yr, and 20-yr phases of future development in addition to existing features. The maps show the general outline of future piers and indicate possible land uses.

These plans were prepared, using the philosophy that the master plan be fixed in basic concept only and that the details be flexible. These plans show no details of the type of structure that might be built on the various land masses; instead, just the land masses are shown with target dates for their construction.

The planning represented on these maps is in accord with the preliminary master plan for the city of Long Beach. It is also coordinated with the citywide traffic planning conducted by the city engineer's office.

Also, if any of the proposed new methods of cargo handling, or perhaps some that have not as yet been visualized, should come into being, the amount of cargo per year over a berth may be greatly increased, thereby reducing the number of berths needed over any given span of years. Because of this, the proposed master plan is made up of increments that are complete entities in themselves and need be built only as they are needed. The basin surrounded by pier F and pier G is such an example.

The development program represented in the master plan maps will make possible the addition of ten berths by 1963, nine more by 1968, and twenty additional berths by 1978.

The basic land areas of these plans, as shown in Fig. 1, are called piers F, G, H, and J. They are designed so that they may be used as open storage areas, traditional transit sheds for break-bulk cargo, combination break-bulk cargo and passenger sheds, roll-on, roll-off facilities, or lift-on, lift-off facilities.

Fig. 2(b) indicates how these various types of facilities may be used in any combination upon a typical pier such as pier G or pier H, depending on how the methods of cargo handling develop.

ECONOMICS OF TYPICAL GENERAL CARGO BERTH

A general cargo berth, as the name implies, may be used for the importation or exportation of a great variety of cargo. A general cargo list would include canned and frozen foods, cotton, lumber, foreign cars, cloth, soaps, animal hides, newsprint, structural steel, and tools. In fact, practically any article produced or used in the United States, except for bulk and liquid products, may be considered as possible general cargo. Because of this wide variety of products, general cargo necessarily comes in various shapes. However, some shipping companies are packaging smaller packages in larger group units by using pallets or "walk-in" containers.

Presently, a general cargo ship is loaded or unloaded out of from one to five of its hatches by ships' gear at an average rate, estimated by various authors and agencies, of from 6 tons to 12 tons per hr per hatch. These are not short period rates, but rates that consider all down time for any reason. At the Port of Long Beach, two 8-hr shifts are ordinarily worked each day. If all five hatches were worked, this would amount to a rate of from 480 tons to 960 tons per day. The average rate in 1956 at all municipal general cargo berths in Long Beach was 940 tons per day. The average general cargo ship that docks in the port of Long Beach loads and unloads a total of 1,880 tons over a 44-hr stay. This 44-hr period is considered to include four 8-hr working shifts.

However, considering 10 berths in 1954 that were handling general cargo but with no steel, foreign cars or lumber, the rates ranged from 397 tons to 1,346 tons per day with an average rate of 771 tons per day. Also on four of these berths, on which the transit sheds had areas of 80,000 sq ft or over, and where the distance on the wharf from pierhead to the face of the transit shed was 50 ft, the average rates exceeded 1,000 tons per day.

An average of sixty-two ships were accommodated at each of these ten berths in the fiscal year 1956-57, and loaded or unloaded an average of 95,000 tons per berth for a total average annual revenue per berth of $76,000.

Of the general cargo shipped through the Port of Long Beach, 52% is foreign, whereas 24% is coastwise, and 24% is inter-coastal. Of this cargo, approximately one-half is inbound and one-half is outbound.

Charts shown on Figs. 10 and 11 indicate tons of cargo per ship and hours in port per ship.

The annual revenue for a general cargo berth is derived from pilotage, dockage, wharfage, storage and demurrage, utility and rental charges. A typical example would be the docking of a C-3 ship. The Harbor Department shares in the pilotage of $0.012 per gross ton for docking the ship and also in the same amount for taking the ship back out to sea. While the ship is at the dock, the charges for tying up are $70.50 per 24-hr day. For every foreign ton of cargo that crosses the pierhead line, the wharfage charges are $0.80 per short ton. These same charges are $0.40 per short ton for coastwise cargo. The shipper has between five and ten days free time, depending upon destination or origin of cargo. If, after this free time, the cargo remains at the berth facility, the shipper must pay demurrage charges that vary from $0.35 per ton per five days to $0.70 per ton per five days.

One of the purposes of a municipal port is to bring business into the area it serves. However, a port should be self-supporting and make a reasonable profit. The average annual revenue per general cargo berth as already shown for the fiscal year 1956-57 was $76,000 per berth. The average annual charges were $66,000. These actual costs were based on general cargo berths operating at about 50% capacity, that seems to be about the greatest efficiency at which a berth can operate, unless it is designed for a particular cargo and is therefore selective. The annual charges for a berth include direct costs, prorated cost, and depreciation. The direct cost includes all items chargeable directly to the berth, such as maintenance and port personnel. The prorated cost includes all port items that should be charged to the berth operations but are not directly chargeable. These prorated charges are charged to the active berths according to their usage.

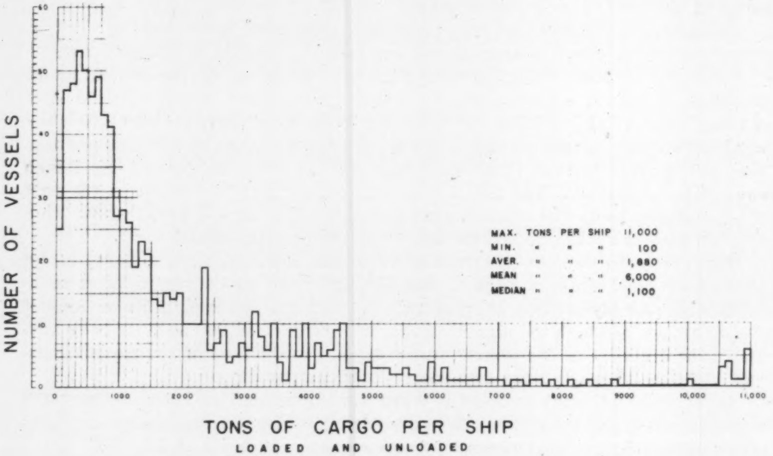


FIG. 10.—GENERAL CARGO TONNAGE PER SHIP AT MUNICIPAL BERTHS, 1956

The estimated cost of constructing a new 600-ft-long, 450-ft-wide general cargo berth, including the dikes, fill, wharf, transit shed, tracks, utilities, pavement, and engineering, is approximately $2,000,000. The rock and sand fill included in this price does not have to be replaced; therefore, the total value of the depreciable facilities is estimated to be approximately $1,600,000. An economic analysis of a typical general cargo berth is as follows:

Actual Revenue - 1956-57 Fiscal Year (Based on 1956 Charges).

Average of 10 General Cargo Berths (95,000 tons per berth)

Annual Revenue		
Pilotage (20% of Total)	$ 1,457	
Dockage	6,881	
Wharfage	53,766	
Storage and Demurrage	10,114	
Utilities	2,584	
Rentals	1,409	
Total		$ 76,200

Annual Costs		
Direct Cost	10,112	
Prorated Port Cost	37,481	
Depreciation	18,755	
Total		66,348
Profit		$ 9,852

Hypothetical Revenue and Cost of a New Optimum Facility.

Construction Cost		
Earth and Rock	$ 373,700	
Concrete Wharf (Prestressed Piles)	517,200	
Transit Shed and Utilities	762,000	
Railroad Tracks	83,200	
Pavement	49,900	
Engineering	178,600	
Total		$ 1,964,600
Less Earth and Rock		373,700
Total of Depreciable Facilities		$ 1,590,900

Annual Revenue (Based on 1959 Charges).—Use 1,000 tons per day times 252 day per year: 50% use = 126,000 tons

Revenue		$ 115,000
Annual Cost		
Direct Cost	$ 19,000	
Prorated Port Cost	44,000	
Depreciation	19,000	
Total		82,000
Profit		$ 33,000

The approach to the depreciation rate on a shipside terminal facility seems to be somewhat a matter of opinion. Some believe that the rate should be a straight line depreciation, based on 75 yr; others believe that it should be straight line, based on 50 yr obsolescence. Another approach is a sinking fund based on government bond interest rates of approximately 2% and based on 50-yr obsolescence.

As the wharf structure and transit sheds are built of concrete and steel and are adequately maintained, the structural life of the facilities may well exceed 50 yr. However, the obsolescence of the structure from the viewpoint of usability probably will not exceed this period. Therefore, the middle of the road sinking fund approach was used, based on 50-yr obsolescence and 2% interest rates.

FUTURE DEVELOPMENT TRENDS FOR A TYPICAL PIER

The general cargo ship replacements for the C series ships are the seafarer, the clipper, and the freedom class of ships. These ships vary little from the C series ships in length, breadth and draft. The largest of these ships, the seafarer, is 529 ft long, 75 ft wide, and has a loaded draft of 30 ft.

Ships for specialized use, such as tankers, are getting larger every year. A recent tanker ship's design was for a 103,000 ton ship that was 900 ft long, 135 ft wide, with a loaded draft of 48 ft. Bulk cargo ships may also approach this size in the future. Other specialized uses, such as trailer ships, will probably be the same general size as general cargo ships because they will be using approximately the same port facilities.

New Type Cargo Ships.—At the present time plans for many new types of cargo ships are being made. The basis of all of the proposed new designs is to reduce ship's port time by revising cargo handling methods such as to reduce manual labor to a minimum and thus greatly reduce the cost of loading and unloading cargo. Although all of these proposed plans and designs are in a formative stage, it is essential that all new plans for port terminals provide facilities that are versatile enough to fit in with any of these proposed ship designs and be so designed as to be able to move cargo to or from shipside at the fast loading and unloading rates that are universal in the proposed designs of all of these ships.

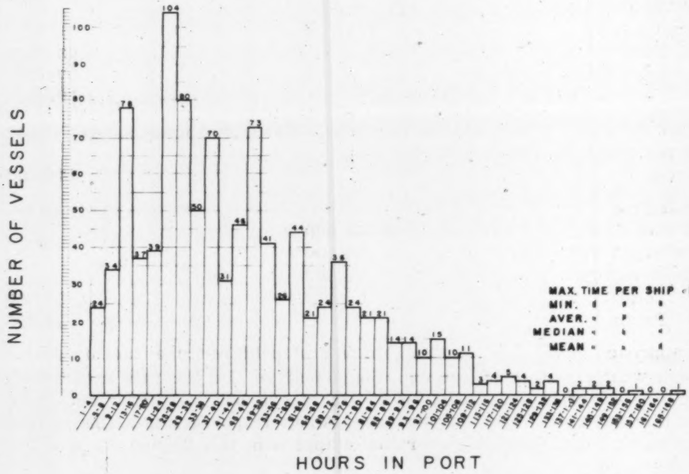


FIG. 11.—CARGO SHIP TIME AT MUNICIPAL BERTHS, 1956

The following data gives some idea of the difference in short period loading rates between present break-bulk type cargo ships and the envisioned ships.

Ship Type	Rate, in tons per hour
Break-Bulk Cargo Ship	150 to 400
Pallet Ship (Sideport-Fork Truck)	350 to 700
Container Ship (Lift-on, Lift-off)	1,200 to 3,000
Trailer Ship (Roll-on, Roll-off)	6,000

As can be seen from the chart, future rates of loading and unloading ships could be from two to fifteen times as fast as present loading and unloading rates.

Fig. 2(a) (typical general cargo covered berths) and Fig. 2(b) (four possible uses of a typical berth) indicate several possible ways of using a typical 450-ft wide by 600-ft long (6.2 acres) berth area. These used are as follows: (1) An open area far handling cargoes such as lumber, foreign cars or sea-vans; (2) open area for "roll-on, roll-off;" (3) covered storage for sea-vans; (4) passenger terminal; and (5) transit shed covered area for general cargo.

Open Area.—The open area provided at the berth provides two wharf tracks, and two low-line tracks for rail transportation. Also provided is an adequate truck loading retaining wall, utilities, lighting and longshoremen facilities.

"Roll-On, Roll-Off."—"Roll-on, roll-off" cargo handling describes a containerized shipping method that allows loaded trucks to drive directly from the pier into the hold of the ship and either leave their trailers to be transported to the desired port or be transported as a complete unit (trailer and truck). This type of cargo handling requires the use of special ships that have loading ramps on both sides and at the stern and have been internally designed to accommodate the trucks with their trailers.

As shown in Fig. 2(b) that depicts the terminal layout for a shipment of "roll-on, roll-off" cargo, the emphasis is on parking space for the trailers. Enough pier space has to be allotted to accommodate both the inbound and outbound shipment.

Sea-Vans.—Reusable containers that are used to transport products from the producer to the consumer via ships are known as "sea-vans." These containers are generally constructed of aluminum or steel, and are equipped with handling rings. The containers may be equipped with refrigeration. Their main advantages are that they reduce handling cost, breakage, and pilferage.

Container capacity usually varies from 10 tons to 15 tons in a range of sizes of approximately 8 ft wide, 8 ft high, and 8 ft to 35 ft long. Cranes used to load or unload these containers may be either shore based on ship-mounted. Shore based units can only be economically justified when there is heavy service between a few ports. Otherwise, ship-mounted cranes will be advantageous.

New container ships are more generally being equipped with ship-mounted cranes. These cranes are of the traveling gantry type with extensible jibs for outreach over the dock, and have a design capacity of 25 tons with a 4 min to 5 min cycle. Each crane has a capacity of 350 to 650 measurement tons per hr.

Shipside facilities for handling sea-vans consist of fork-lift trucks and straddle trucks, each capable of handling the individual containers. Open-sided transit sheds, wide-apron wharves, and special loading ramps are also desirable, see Fig. 2(b).

Passenger Terminal.—A passenger-cargo transit building, as visualized, should be a three-level dual operational facility, efficiently and comfortably handling both passenger and cargo traffic. The lower level or ground floor is entirely for cargo movement and storage, and in general will be a typical transit shed plan. The second level will be the basic approach, circulation, and parking facility for vehicular transportation of tourists, visitors, and luggage for passenger liners. Vehicles enter and leave this second level by means of an elevated access roadway ramping from the main thoroughfare. Thus, cargo handling facilities of the ground floor will be completely separated from the passenger transportation traffic. Additionally, the second level provides the efficiency and convenience of immediately adjacent driveway and parking facilities for passenger-and-luggage transit operations. Vehicular traffic areas should be ventilated by decorative grilles between columns or otherwise at the exterior walls.

The second level will also contain a large screened area allocated for luggage transfer and inspection by United States Customs. The area can be immediately adjacent to a long passenger-and-luggage traffic island, thus providing an efficient and convenient baggage transfer system. Office accommodations are also desirable at one end of this second floor.

Vehicular ramps and escalators should lead from the second floor upwards to a third level providing comfortable transition for passengers and visitors. The third level should be nearly level with the promenade or "A" deck of passenger vessels for comfortable gangway access. The landside longitudinal half of the third level could provide additional driveway and parking facilities. Touring buses and taxis can stop at an extended loading island, providing convenient access accommodations for transient passengers of incoming vessels. The waterside longitudinal area should offer a sightseeing promenade balcony the full length of the building and various reception salons, foyers, waiting rooms, escalators, and accommodations arranged to provide the maximum convenience for passengers and visitors. A large restaurant may be incorporated at one end of the upper level, offering a spectacular high-level view of harbor activities through window-walls. Office areas can also be provided at the ends of the third level.

In brief summation, the contemplated passenger and cargo transit building superimposes passenger vessel accommodations above a typical transit shed floor plan, combining but separating these two different operations. In addition to isolating the activities, the superposition scheme raises the floor level of the passenger facilities to similar upper deck heights of the passenger ships for horizontal gangway access.

General Cargo Berth.—Using the C-3 as typical of the American ships in use today, and the seafarer class ship as typical of those in future use, minimum berthing standards can be established for a general cargo berth. The ship berth should be 600 ft long and should have a depth alongside of 35 ft at mean lower low water. The minimum distance between piers should be 400 ft to allow for the ships at either pier, bunkering and ship movement in between. (See Figs. 2(a) and 8).

Alongside the berth there should be a covered transit storage area. The Port of New York Authority, using a typical general ship cargo of 12,500 measurement tons half discharged and half loaded, and assuming 70 cu ft per measurement (revenue) ton, 14-ft stack height, 25% waste space, and 50% working aisles, estimates[4,5] the need of 90,000 sq ft for this area. Kenneth Peel in his examination of transit shed design,[6] recommended 100,000 sq ft per berth for transit storage. His figure was based on a ship with 770,000 cu ft of cargo space, 12-ft stack height, and 40% wasted space.

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[4] "Modern Design of General Cargo Marine Terminals," by Frank W. Herring, Planning Div., Port of New York Authority, October 19, 1954.

[5] "Increasing Port Efficiency," Journal, ICHCA (New York Symposium, Pace College), March, 1957.

[6] "Transit Shed Design," by Kenneth Peel, Instruction for Harbor Design Course, Univ. of California Extension, Fall, 1951.

Using the actual Port of Long Beach figures of approximately 2,000 tons per ship, two days per ship, maximum free time, 20-ft stack height, and 40% wasted transit shed space, the area of transit shed needed to keep a ship at the berth 100% of the time is estimated at 93,300 sq ft.

From these rationalizations, it would appear that a logical transit shed area should, therefore, be at least approximately 90,000 sq ft. However, experience has shown that transit sheds containing 80,000 sq ft or more, have been adequate for cargo unloading rates of 1,000 tons per day and more. As shown in Figs. 2(a) and 8, the transit sheds have 80,000 sq ft per berth (160 ft wide and 500 ft long).

In order to handle large units of cargo and the resulting equipment, the transit sheds should have a minimum eave height of 20 ft, and the entire waterside of the shed, except for columns, should open. The minimum door height should be 18 ft. The use of as few interior columns as possible helps the flow of goods.

All ramps, transit shed openings, wharf load limits, and back area layouts should be designed to facilitate present and future materials handling techniques and equipment.

The entire pier area should be paved. Asphalt pavement has proven satisfactory for both the open and covered areas.

Adequate lighting should be provided for 24-hr use of the entire pier. Also, in order to provide low fire insurance, fire resistant construction, and fire protection equipment such as automatic sprinklers in buildings, fire hydrants, and fireboats should be provided.

The transit shed should be set back from the pierhead line about 50 ft in order to allow a wharf space for loading and unloading cargo. This 50-ft width allows for the installation of as many as three wharf tracks or two wharf tracks and a crane rail. However, for general cargo purposes, the use of one or two wharf tracks is sufficient. The wharf should be designed for a minimum surcharge of 750 psf. This loading allows for full flexibility in the use of the wharf area for cranes, trucks and trains.

In order to adequately hold the ship to the berth, bollards and cleats should be alternately spaced along the pierhead line about 65 ft to 75 ft apart. To supply the utility requirements of the ship, two water outlets two inches in diameter protected with back flow preventors, one combination 208-120 v power outlet, and one telephone outlet should also be provided at each berth.

It is estimated that over two-thirds of the land transportation to a general cargo berth is furnished by trucks. A marginal type wharf where a loading platform extends the full length of transit shed will provide approximately forty truck stalls.

The average truck load is 10 measurement (revenue) tons. It takes about 2 hr on the average to load or unload a truck. On the basis of an 8-hr day, forty truck stalls will handle one hundred sixty trucks per day (1,600 measurement tons of cargo). Other trucks could also be loaded or unloaded by the use of fork lifts.

Rail transportation to and at the berth, although decreasing in volume, is still essential. Bulk commodities and heavy loads are still transported almost exclusively by rail. Also, in times of national emergency, rail transportation gains added importance. Trackage needed at a general cargo berth includes one through track and at least one side track on the wharf, and at least one track at the loading platform. Back area tracks should provide adequate loops for efficient train movements, and adequate storage.

Back area storage space, either covered or uncovered, should be provided for each berth. This space should roughly equal one-half the transit shed space (40,000 sq ft). Lumber, cotton, structural steel, foreign cars, and sea-vans are examples of cargo requiring this back area. Some of these cargoes, such as cotton, require the use of warehouses or covered storage.

The pier should also provide adequate space for parking for longshoremen's cars, offices for shippers, terminal operator facilities and restaurants. Parking facilities for approximately 125 cars per berth should be provided. If space limits are critical, multiple story parking garages could be built. Each shipper and terminal operator usually requires office space at the berth. Terminal operation (stevedoring) facilities should be built on the approximate basis of one facility for five berths. A few coffee shop type restaurants should be provided in the entire harbor area.

CONCLUSIONS

The comparatively rapid growth of the Port of Long Beach is guided by a long-range master plan. The purpose of this plan is to provide an efficient and self-sustaining port which will meet the present and future needs of the surrounding area and its hinterland.

The preparation of this master plan has required the development of certain basic planning and design criteria for port structures that, in general, are as follows:

1. Flexible pier layouts that can be utilized for various cargoes and which allow a minimum of 6 acres and an optimum of 8 acres of area per berth;
2. 600-ft long berths;
3. 35 ft minimum water depth at pierhead line;
4. 50-ft wide waterside apron wharves with tracks;
5. 80,000 sq ft minimum area clear span transit sheds;
6. Loading platforms with adjacent tracks; and
7. Paved back areas for warehousing, open storage, and automobile parking.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3244

SAN DIEGO TRANSPORTATION PLANNING STUDIES

By Edward M. Hall,[1] M. ASCE

SYNOPSIS

San Diego, Calif. has established a continuing urban transportation planning program. The concepts and administrative organization of this metropolitan area transportation study are described. Selected examples demonstrate the wide range of factual approaches to a coordinated solution of transportation problems. The importance of all levels of government and agencies concerned working together is emphasized.

INTRODUCTION

Who is responsible for transportation planning in metropolitan areas (the state, the county, the region or metropolitan area, or the city)? What department of the city is responsible for urban transportation planning (planning, engineering, public works, management, or police traffic)? The answer is simple and clear. All levels of government and all departments and agencies concerned must do the job together.

The American Municipal Association.—American Association of State Highways Officials (AMA-AASHO) Joint Committee on Highways and the Urban Research Committee of the Highway Research Board in cooperation with Syracuse University sponsored a National Conference on Highways and Urban Development in October, 1958. The Sagamore Conference emphasized cooperation and

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[1] Transp. Research Dir., San Diego, Calif.

coordination of the urban transportation challenge and did much to define the areas of responsibilities for action.

SAN DIEGO PROGRAM

In 1954, the city of San Diego, Calif. reached a decision to implement a program for the orderly accumulation of facts and to coordinate the application of these facts toward the solution of growing urban transportation problems. Following several meetings with political and public support leaders in the metropolitan area which indicated widespread approval for an urban transportation program, a series of conferences was held to develop a technical approach to the program. Representatives of the Automotive Safety Foundation, the Bureau of Public Roads (Department of Commerce - BPR), the California Division of Highways, the Automobile Club of Southern California, and the Institute of Transportation and Traffic Engineering of the University of California conferred during a two-day period and put together the basic ideas and objectives of the San Diego Transportation Studies.

These guiding principles are as follows:

1. Urban transportation planning must be a continuing function of government rather than a "one-shot" approach.
2. Transportation problems of a metropolitan region have no jurisdictional boundaries; therefore, any study must include the entire San Diego metropolitan area and all jurisdictions or agencies concerned with transportation.
3. The study must concern itself with all types of transportation utilizing streets and highways, transit, the terminal requirements, and with related functions such as administration, laws, and financing.
4. A small nucleus coordinating staff should be created to work with and through the existing departments. This is, perhaps, the most important principle of all. Included in this concept was the thought that existing departments should be augmented to conduct such additional studies as might be required during the transportation study and that the basic responsibilities of the department must be maintained and their knowledge and experience utilized.

ORGANIZATION

The Problem.—The basic guiding principles presented a challenging problem in administrative organization. It was felt that if the technical staff people of the city could be brought together, the sincere differences that might occur between departments would be resolved.

The lack of a formal metropolitan or regional government presented another challenge. How can seven individual cities, the county, the state Division of Highways, a private transit system, and the United States Navy be formed into a technical working team? San Diego's answer is as follows:

History.—On March 1, 1955 the Office of Transportation Research was established within the Department of Administrative Management as a part of the City Manager's staff. Transportation Research Staff consists of a director, a staff engineer, a staff planner, a draftsman, and a secretary.

Advance Transportation Planning Team.—Shortly after the formation of the Office of Transportation Research the idea of a new type of administrative organization was conceived. This idea has resulted in one of the most interesting accomplishments, the organization of an Advance Transportation Planning Team for the San Diego Metropolitan Area. This team is composed of five people representing the City Planning Department, the City Highway Design Engineer, the City Traffic Engineer, County Planning Department, and the County Surveyor and Road Commissioner. These five technicians work together as a full-time team in one office separate from day-to-day operations. The members of the team work under the guiding supervision of the Transportation Research Director, but report directly to the department heads of the city and county whom they represent. In this manner they act as liaison officer to the department heads keeping them informed of what is going on in the Advance Transportation Planning Team and in Transportation Research, and bringing to the team and to Transportation Research the data, knowledge, and day-to-day activities of their respective departments. This procedure had the additional important advantage of maintaining the proper balance between operational problems and long-range thinking. Its function is to serve as a two-way street of communications.

Through the Advance Transportation Planning Team three elements of transportation planning are brought together with an appreciation of each other's concepts and problems. As the city team reaches a recommendation concerning a study or special assignment, it is presented to a meeting of the city department heads. The department head then has an opportunity to present his ideas through his representative and then has the further opportunity to discuss the problem with other department heads concerned. It has been found that the joint conclusions of the competent technicians working together on the team are impressive.

Technical Coordinating Committee.—From its inception the San Diego Transportation Study has been based on the premise that transportation is a metropolitan area problem and must be studied as such. To coordinate this metropolitan area study a Technical Coordinating Committee was organized. It is composed of the planning directors, city engineers and traffic engineers of the seven cities, the county, and the California Division of Highways working together on an informal basis. Representatives of the San Diego Transit System, Navy, Traffic Division of the San Diego Police Department, and the Industrial Coordinator and Urban Renewal Coordinator are also members of the Committee. This group has met once a month since May, 1955 to review the progress of the study and to discuss general policy in an advisory capacity. The Technical Coordinating Committee has substantially eliminated, at the technical level, the problems that often occur between a large central city and the small individual communities nearby. A common understanding and a spirit of cooperation has prevailed within this committee from its beginning.

Executive Committee.—As this transportation study has shifted from its initial fact-gathering phases to the application phases, it has become increasingly necessary to have a small steering and implementing group. The Executive Committee was formed in the fall of 1958 and consists of the Assistant City Manager as Chairman, the Director of Public Works-Water-Utilities and Engineering, the City Engineer, the City Planning Director, and the Transportation Research Director who serves as Secretary. This five man Executive Committee has provided the means for implementing the results of several urban transportation planning studies. Further, it furnishes a means for focusing attention on certain administrative problems and thereby finding early solutions to such problems.

This committee is organized on an informal basis. The District Engineer of the California Division of Highways, or the County Engineer, or the County Planner, or a Transit System official can be invited to attend at such times as it is important to have their counsel on the committee.

PILOT CITY PROGRAM

During the same period that the San Diego Metropolitan Area Transportation Study was developing, the National Committee on Urban Transportation was formed. This committee prepared procedural manuals to establish the uniform urban transportation fact-gathering procedures, and selected pilot cities in March, 1956 and has undertaken to field test all the nine areas of the National Committee draft manual. In January, 1958 San Diego submitted an extensive report on its work during the pilot city program to the National Committee.

A great deal has been learned in the process of field testing these manuals. Because San Diego had a year's head start in getting its local organization underway, it was able to make many contributions in the preparation of these manuals through the process of reviewing the results of the practical application. San Diego has materially benefited from the program by amassing a large amount of factual data, using this data in day-to-day operations, and most important, applying the data, techniques and concepts that have been developed to long-range urban transportation planning.

In its guide manual,[2] along with the series of procedural manuals, the National Committee on Urban Transportation has presented the efforts of a hundred and seventy-five leading people. These works are designed to help cities and metropolitan areas in the solution of urban transportation problems. If San Diego's experience is typical, the guide and the manuals will be extremely useful to cities in all population groups attempting to solve their transportation problems and to coordinate transportation planning activities with other jurisdictions and agencies.

SELECTED EXAMPLES OF TRANSPORTATION PLANNING

The functioning of the administrative organization for a program of urban transportation planning is illustrated by briefly describing five selected examples that involve the application of the factual approach to the coordinated solution of transportation problems in a metropolitan area.

Travel Time and Level of Service.—The quality of urban traffic service may well be measured by three criteria: first, travel time in terms of a delay rate; second, safety in terms of an accident rate; and third, economics (the cost and benefit of any given level of service). The latter may well be expanded to include the economic relationships between transportation and the total economic balance of a region.

Travel time studies have been completed on 346 miles of urban streets and highways. Fig. 1 shows 273 miles of freeway, major, collector and local streets during 1957. The San Diego Transit System has participated by conducting transit travel-time studies on a number of routes.

These travel-time studies have been related to the functional concept of street classification. This concept holds that a street system has two basic purposes which are diametrically opposed. These basic purposes are to move traffic and to provide access to abutting property. The San Diego travel-time studies have developed levels of service in terms of speeds which are practical of attainment on the various geometric designs related to the different functional classifications of streets and highways. Table 1 shows the level of service their studies have developed for the various functional street classifications in terms of average overall speed.

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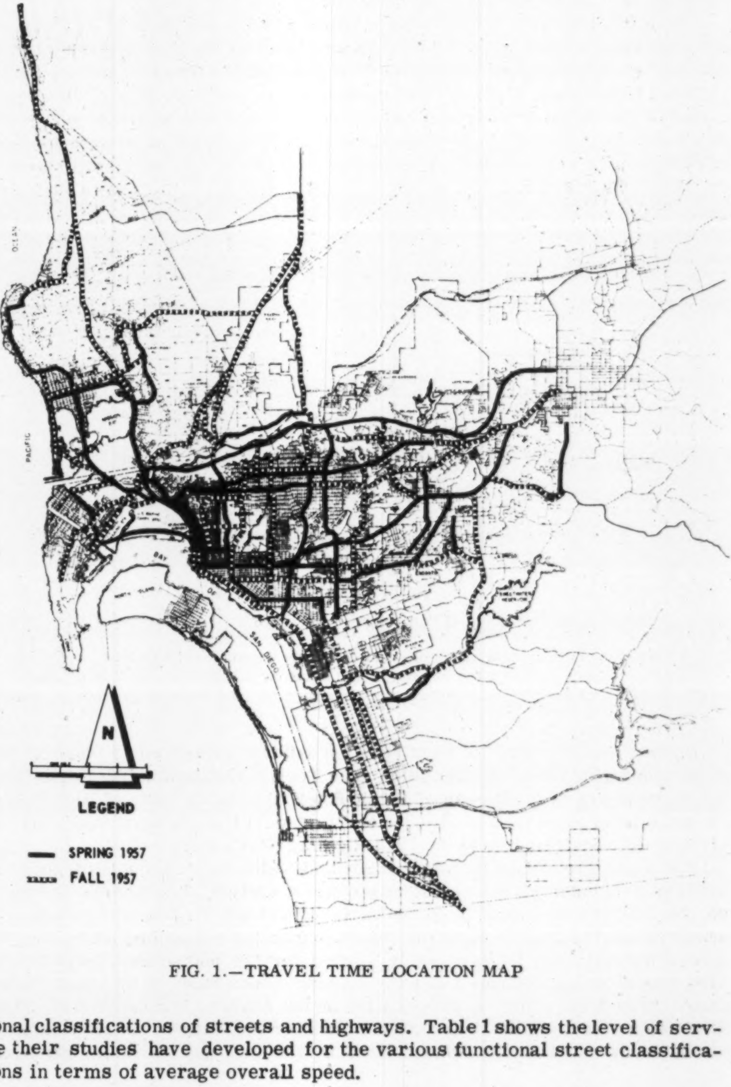
[2] "Better Transportation For Your City," Natl. Committee on Urban Transp.


OCEAN
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SAN DIEGO
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LEGEND
SPRING 1957
FALL 1957

FIG. 1.—TRAVEL TIME LOCATION MAP

The travel-time studies enable the presentation of a simple measure of the existing level of street and highway transportation service. Fig. 2 shows the existing level of service in San Diego in terms of equal time contours radiating out from the Civic Center. This analysis clearly demonstrates that present traffic facilities enable the motorist to travel 6 miles farther in a north-south direction than in an east-west direction in 25 min.

The level of service concept also provides a tool to evaluate the city's future freeway system plan. Fig. 3 portrays the level of service that our planned freeway system would provide for the "horizon year" in terms of the population included within radii enclosing one million and two million people. The estimates of the city and county planning departments forecast a horizon year population of 2.5 million for the metropolitan area. This assumes present concepts of development will continue. This evaluation concept was developed based on the idea of the historical 30 min city - travel time from home to work 30 min or less.

TABLE 1.—TRAFFIC ASSIGNMENT OVERALL SPEEDS

Classification (1)	Overall Speed, in miles per hour	
	Urban (2)	Rural (3)
Freeway[a]	50	55
Expressway[b]	35	50
Major Highway	—	50
Major Street -		
Normal	30	—
Highly developed area	20	—
Feeder	25	40
Local	15	—
Business Street (C. B. D.)	15	—

[a] Freeway — Full control of access and complete separation of grades.
[b] Expressway — Full control of access and some or all intersections at grade.

Highway Needs.—The 1971 Highway Needs Study presented an ideal project to be undertaken by the metropolitan area Technical Coordinating Committee. Representatives of the Bureau of Public Roads and the Division of Highways at the state level came to a meeting of the Technical Coordinating Committee to present the requirements of the 1971 Highway Needs Study.

The committee undertook this study and, using uniform design standards, developed together a freeway and major street system. The system was based on the land use and population estimates developed by the several planning agencies and the traffic estimates resulting from these planning concepts. The street, highway, and freeway study system for the 500 sq mile metropolitan area was in turn coordinated with the Highway Needs Study of the county of San Diego through the county representative on the Advance Transportation Planning Team. Perhaps the degree of coordination achieved in the 1971 Highway Needs Study can best be demonstrated by the fact that the seven cities, the county, and the Division of Highways all submitted the same identical system map for the San Diego metropolitan area.

Freeway System.—The 1971 Highway Needs Study naturally led into necessary studies for one of the outstanding pieces of highway legislation in California in recent years. Senate Concurrent Resolution No. 26 adopted by the 1957 legislature required a state-wide study of ultimate freeway and expressway system for the state of California and for the connections thereto, irre-

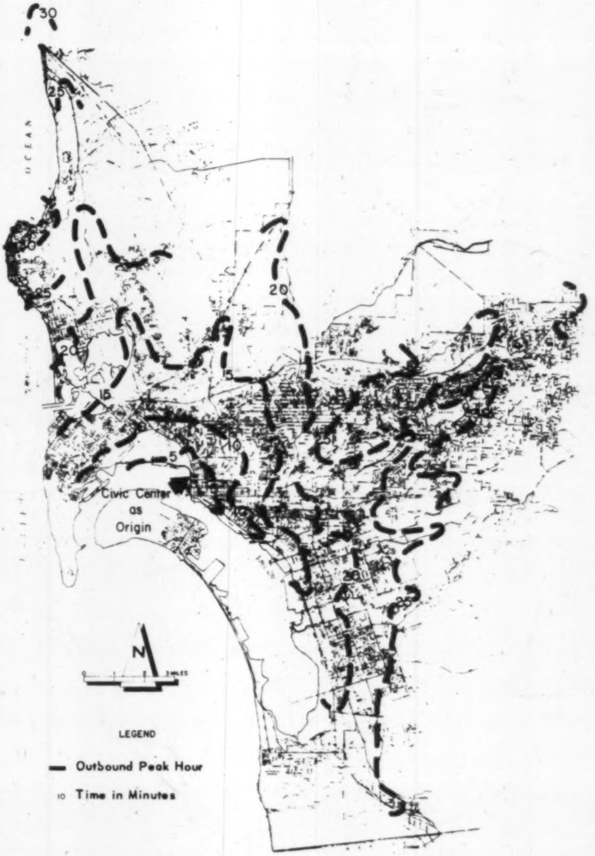
30
25
OCEAN
25
20
20
15
10
5
Civic Center as Origin
20
25
N
0 1 2 3 MILES
LEGEND
▬ Outbound Peak Hour
10 Time in Minutes

FIG. 2.—TIME CONTOUR MAP: OUTBOUND PEAK HOUR

spective of jurisdictions. Again, the Technical Coordinating Committee served to bring together the knowledge, data, and concepts of the several jurisdictions in the technical fields concerned in such an important long-range study.

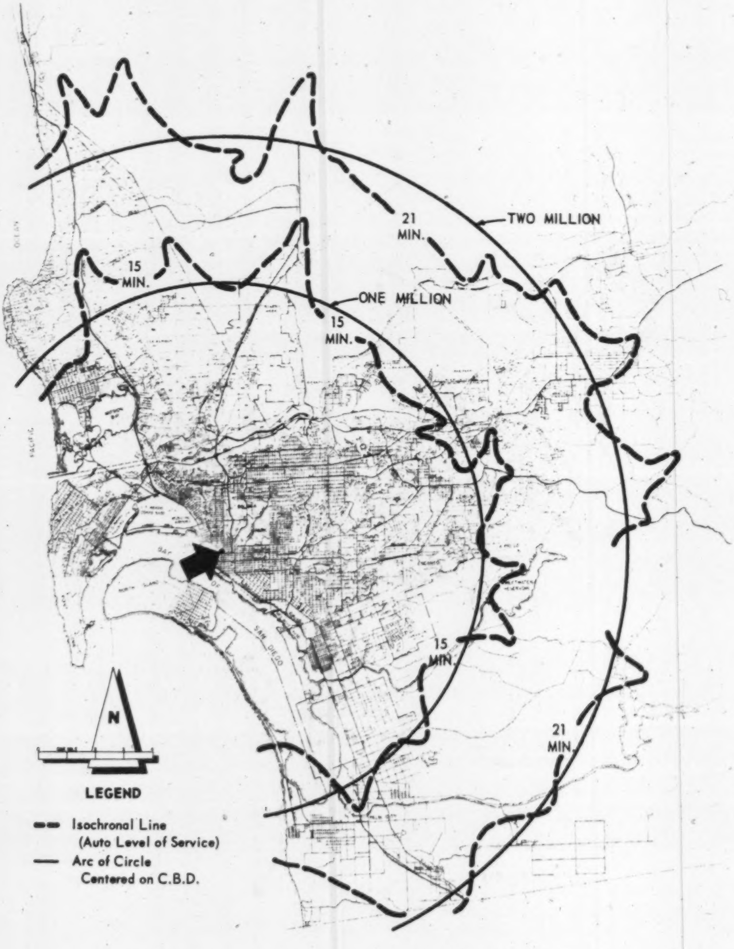
21 MIN.
TWO MILLION
15 MIN.
ONE MILLION
15 MIN.
15 MIN.
21 MIN.
SAN DIEGO
N
LEGEND
Isochronal Line (Auto Level of Service)
Arc of Circle Centered on C.B.D.

FIG. 3.—AREA ENCOMPASSING ONE AND TWO MILLION PEOPLE

The city and county planners developed an estimate of future land use for the entire metropolitan area based on land-use surveys and analysis of the holding capacity of the land. Planners further developed population estimates for the horizon year as well as for intervening target years.

City Traffic Engineering and Transportation Research developed studies of the traffic generation and attraction of various land uses. These studies were based on the following basic studies:

1. Origin-destination studies of the metropolitan area conducted by the California Division of Highways in cooperation with the Bureau of Public Roads in 1952-53.
2. Intensive study of two large residential sub-divisions made during the summer of 1956 by the city of San Diego.
3. A number of specific studies of industrial and commercial areas and communities.

During this period, city, county, and state highway design engineers were studying the costs and practicality of alternate routes for freeways and major streets and highways within the 500 sq mile study area.

The California Division of Highways developed the traffic assignment procedures and the machine processing, using electronic computers, to produce traffic estimates on the study plan of freeways-expressways, major streets and highways. This assignment procedure used traffic generation estimates based on land use characteristics and relationships and was distributed by gravitational formula based on the interrelation of land use and travel habits.

The cooperation during this study was to the extent that personnel were actually traded between the Division of Highways and the city of San Diego when different skills were needed to assist in different phases of the operations. There were constant meetings and a division of work assignments between the major jurisdictions, (state, county, and city of San Diego).

A state-wide system of freeways has been developed by the Department of Public Works of the State of California for submission to the State Legislature. In San Diego, the traffic assignment was used to evaluate the study plan. Based on the traffic assignments to the initial study plan, a slightly modified study plan was developed for SCR 26. The SCR 26 freeway system is shown in Fig. 4.

San Diego then engaged on the assignment of traffic to the SCR 26 Freeway System for the horizon year; that year when the population of the San Diego metropolitan area will equal 2.5 million, and for the year when the population will equal 2.0 million. Further, the basic data for traffic assignment for the years when the population will equal 1.5 to 1.0 million people, will be developed. This will facilitate the development of both a plan and program for the staging of a completely integrated and properly balanced freeway system.

Parking Needs.—In an effort to develop an estimate of the parking needs to correspond to the 1971 Highway Needs Study, the city's parking requirements were projected to 1971. Study of usage, duration, and occupancy, both street and off-street, were made. These studies were basically sampling studies to determine overall planning data for the central district of the city of San Diego.

Fig. 5 shows the historical parking supply data. Further, Fig. 5 visually presents a summary of the estimated parking needs of 1971 for the various areas of the central city. The parking needs cost estimates are summarized in Table 2. It would appear that the parking needs for the central district of San Diego can be provided at an average expenditure of approximately $1,000,000 a year over the 15 yr period.

Encanto Study.—In addition to the overall planning studies the factual approach has also been applied to specific major street and specific freeway location studies. An example concerned the area of Encanto in San Diego. This study was made in 1958. The problem was to determine which of two principal freeway locations would best serve existing and future estimated traffic and have the least adverse effect on developed and developing property.

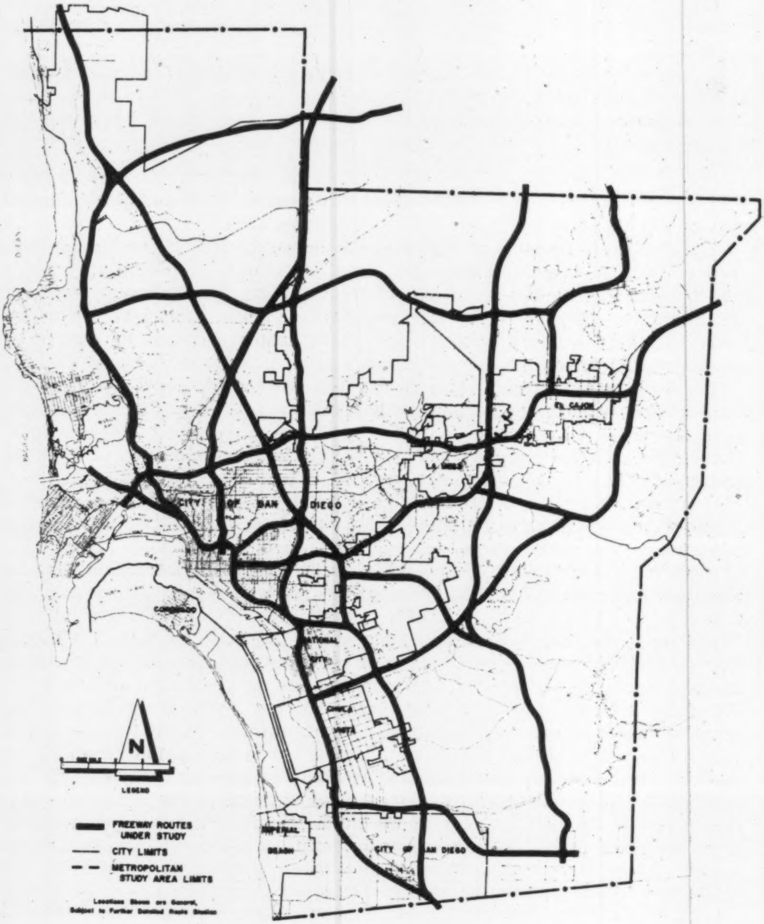
CITY OF SAN DIEGO
EL CAJON
LA MESA
CORONADO
NATIONAL CITY
CHULA VISTA
IMPERIAL BEACH
CITY OF SAN DIEGO
N
LEGEND
FREEWAY ROUTES UNDER STUDY
CITY LIMITS
METROPOLITAN STUDY AREA LIMITS
Locations Shown are General, Subject to Further Detailed Route Studies

FIG. 4.—SCR 26 FREEWAY SYSTEM

Studies were made of existing development and the subdivision activities during the several preceding years. Additional subdivisions in the near future were indicated by tentative subdivision maps on file. Further studies included

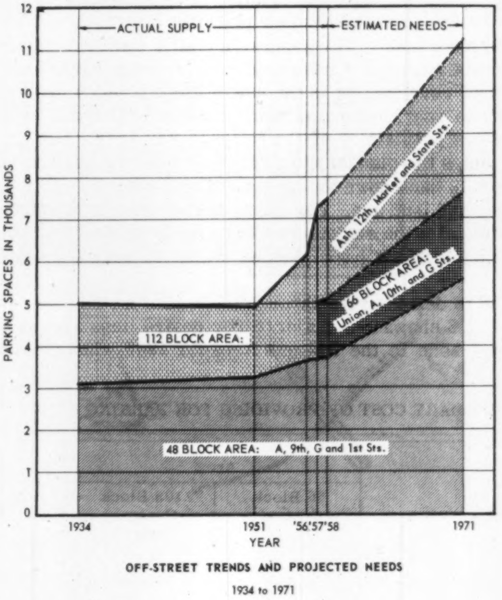

FIG. 5.—ESTIMATE OF PARKING NEEDS: 1971

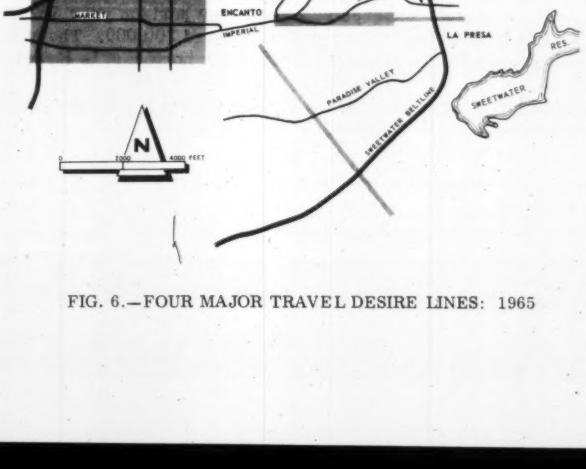
BROADWAY
LEMON GROVE
1965
WABASH
STATE HIGHWAY 94
MADERA
MARKET
ENCANTO
JAMACHA
IMPERIAL
LA PRESA
RES.
SWEETWATER
PARADISE VALLEY
SWEETWATER BELTLINE
N
0
2000
4000 FEET

FIG. 6.—FOUR MAJOR TRAVEL DESIRE LINES: 1965

the holding capacity of the land and location of existing and proposed school and park sites. Based on these studies planning forecasts and estimates of traffic generation were made. The generated traffic was then distributed over the various alternate systems. These studies were made for the horizon year. Then, through a new development in the use of factors which are growth fractions of the horizon year, estimates were made for the traffic that would be expected for various populations of the metropolitan area. The growth fraction technique enables the planners to first determine the horizon year traffic estimates and then to back down the interzonal transfers by using parameters such as population and land use. For the traffic experts it is emphasized that this technique eliminates the serious problems of the zero interchange and the infinite growth factor which have plagued traffic forecasting analysis for so long.

It was possible to demonstrate, through the traffic model, the reorientation of traffic flow. Fig. 6 shows four major traffic desire lines radiating from one traffic assignment zone in the Encanto Area for 1965, when the metropolitan area population would be 1.0 million. Fig. 7 shows the growth and orientation of the same four general traffic desire lines for a population of 1.5 million, 1980. Fig. 8 shows the overall growth and the reorientation at the horizon year. It was felt that this mathematical model of traffic orientation or distribution was quite logical when it was borne in mind that the area to the southeast is, in 1959 completely undeveloped, but in the horizon year would be expected to have a population in the neighborhood of 200,000. The area to the east is developing rapidly now and the area to the northeast is largely developed.

TABLE 2.—SUMMARY COST OF PROVIDING FOR PARKING NEEDS - 1971

Item (1)	Area: 66 Block Central Core (2)	Area: 108 Block Central District (3)	Total (4)
Spaces	2,430	1,480	3,910
Unit cost per stall	$3,840	$3,520	——
Total cost, millions	$9.3	$5.2	$14.5
No. projected years	15	14	——
Cost per year, thousands	$622	$372	$994

Fig. 9 summarizes the traffic forecasts which clearly indicate several important conclusions:

1. A freeway is not required in this area prior to 1965.
2. A freeway is required in the vicinity of 1980 or when the metropolitan population reaches approximately 1,500,000.
3. A major street will adequately serve the interim and long-range needs in a direct east-west line.

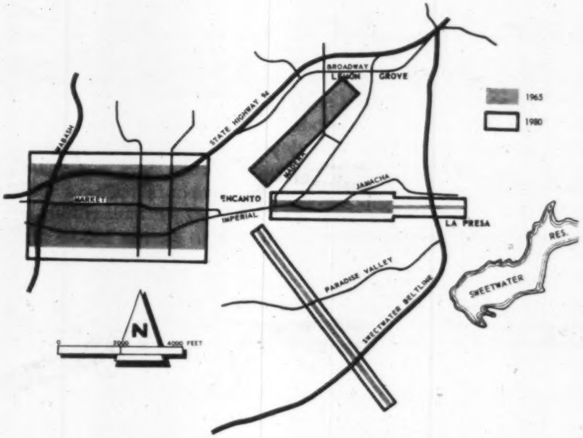
BROADWAY
LEMON GROVE
STATE HIGHWAY 94
WABASH
MARKET
ENCANTO
IMPERIAL
JAMACHA
LA PRESA
PARADISE VALLEY
SWEETWATER BELTLINE
SWEETWATER
RES.
N
0
2000
4000 FEET
1965
1980

FIG. 7.—FOUR MAJOR TRAVEL DESIRE LINES: 1980

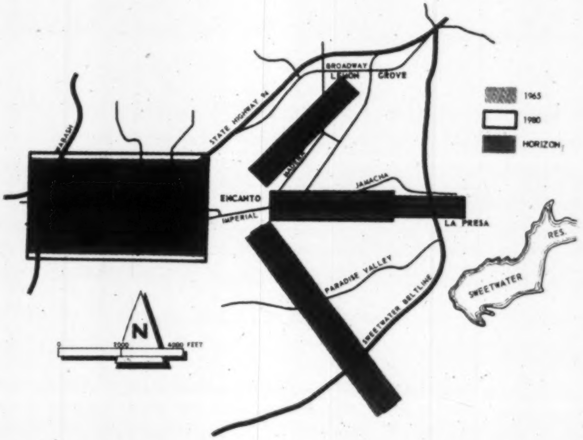
BROADWAY
LEMON GROVE
STATE HIGHWAY 94
WABASH
ENCANTO
IMPERIAL
JAMACHA
LA PRESA
PARADISE VALLEY
SWEETWATER BELTLINE
SWEETWATER
RES.
N
0
2000
4000 FEET
1965
1980
HORIZON

FIG. 8.—FOUR MAJOR TRAVEL DESIRE LINES:HORIZON YEAR

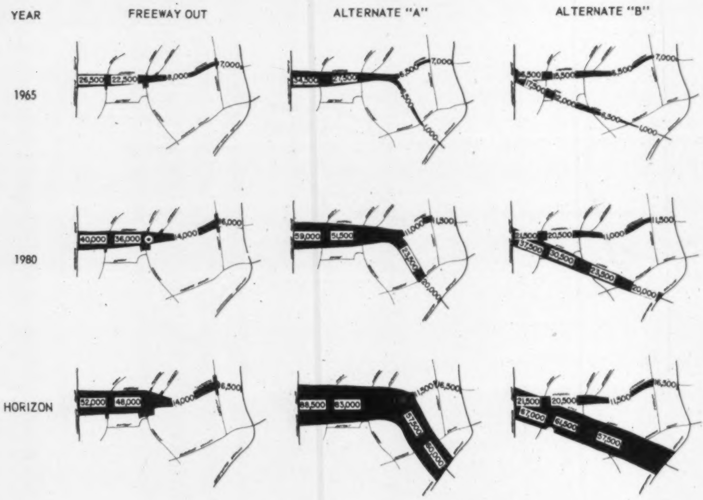
YEAR
FREEWAY OUT
ALTERNATE "A"
ALTERNATE "B"
1965
26,500
22,500
8,000
7,000
27,500
6,500
7,000
1,000
6,500
16,500
8,500
7,000
17,500
6,500
1,000
1980
40,000
36,000
14,000
16,000
59,000
51,500
11,000
11,500
23,500
20,000
21,500
20,500
11,000
11,500
37,500
30,500
23,500
20,000
HORIZON
52,000
48,000
14,000
16,500
88,500
83,000
11,500
16,500
57,500
50,000
21,500
20,500
11,500
16,500
67,000
61,500
57,500

FIG. 9.—ESTIMATED TRAFFIC VOLUME, ENCANTO STUDY

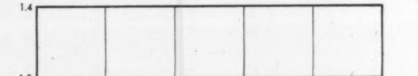

FIG. 10.—METHOD OF COMPUTING THE ECONOMIC BREAKPOINT FOR ALTERNATE FREEWAY LOCATIONS

Engineering design and economic studies were conducted to enable the computation of the costs and benefits likely to be expected from the principal alternate freeway locations. Fig. 10 demonstrates a method for computing the economic breakpoint and thus the most economical traffic service location. Line A plots the percentage (50%) the minor movement is of the total traffic in 1965. Lines B and C plot the same relation (32% and 17%) for 1980 and horizon year, respectively. Point I is the time and distance balance point between the travel time savings of the minor movement and the adverse distance introduced on the Main Line. The conclusion is reached that the location should serve the horizon year major movement if constructed any time after 1970, since a 20-yr amortization period is used. This and the traffic data enabled the conclusive determination that a freeway was not needed in the Encanto area in 1959 and would not be needed until nearly 1980. Then it would most economically be located on the southerly location.

In addition to the answers furnished by the planning data, traffic forecasts, design and economic studies, a guiding tool was developed for estimating how far in advance it was economically justifiable to acquire rights-of-way. Although preliminary, this economic slide rule type device has clearly indicated generalized answers.

These studies thus demonstrated factually the type of facility required, the best location for the freeway, the necessary level of traffic service, the most economical service to traffic, and the minimum adverse effect on land and its development.

Planning A Balanced Transportation System.—Selected studies that have been completed in the San Diego metropolitan area have been briefly outlined. In addition, a large number of fact-gathering projects have been implemented on a continuing basis. Large quantities of factual data have been obtained and analyzed and are being applied in our urban transportation planning studies. A soundly developed freeway system exists which is being refined by additional study and analysis. It is felt that an adequate estimate of the parking needs for the central district of the San Diego has been prepared.

However, it is emphasized that the city is striving to be able to integrate and balance the total land transportation system of highways, major streets, terminal facilities of all kinds, transit, and trucking. A balanced transportation system will undoubtedly have varying degrees of use of travel modes in different areas throughout the metropolitan region. For example, in the downtown area we can expect high percentage of transit use, whereas the outlying areas in the horizontally developed region such as San Diego a minimum transit use can be expected. This does not mean that transit is unimportant or should be excluded from our planning. In fact, the converse is surely true. For instance in San Diego, 35% of the peak hour people entering and leaving the central district are moved by transit. Think what this would mean if it were necessary to move an additional 10,000 people in approximately 6,700 cars on the existing street and freeway system in the peak hour.

The city is searching for a means of studying the economic balance of the total transportation system in relation to the level of service in terms of travel time and safety. There is hope that during 1960 inroads can be made into this challenging problem of evaluating the balanced transportation system.

The importance of achieving a balanced transportation system is emphasized by the large sums of money being invested in San Diego urban areas and particularly the total for the metropolitan region. In San Diego it has been estimated that to provide adequate street and highway transportation service by 1971, it will require a total investment of public and private funds of 1.8 billion dollars. In addition to that, it has been estimated that the private vehicle terminal facilities of the downtown area of San Diego will require the investment of another fifteen million dollars. There is a real need to study these costs in relation to the cost of providing convenient and efficient transit and to relate the theoretical capacity and efficiency of transit and to the desires of the people. In summary the effort is to obtain the best balanced system which is acceptable to the public, will be used, and is financially feasible.

CONCLUSIONS

The challenging importance of the Interstate Highway Program and the cost of this program and the current economic problems facing the United States emphasize the need for cooperation and coordination both within and between jurisdictions. As the result of the transportation program and the pilot city studies, San Diego is beginning to accumulate a fund of basic factual information which enables the city more adequately to define and measure urban transportation problems. It has been found that the Advance Transportation Planning Team and the Technical Coordinating Committee provide the tools to effectively and intelligently solve together the complex technical problems involved in the planning of the interstate, the freeway, and the major street system of a metropolitan area. It is imperative that technical people approach the job in a spirit of full cooperation. It is San Diego's experience that this approach works. It has resulted in public confidence in the joint planning effort as applied to the highway program. This is demonstrated by the fact that nearly all the city's freeway route adoptions and freeway agreements are approved with little or no controversy.

In San Diego the objective was to establish the function of urban transportation planning on a continuing basis commensurate with the needs and growing importance of moving people and goods. Only through the continuing effort of cooperative transportation planning, coordinated with one goal in mind and with the maximum implementation of the Federal highway program, will local governments be able to resolve their urban transportation problems.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 3254

ROADBEDS ON HIGHWAYS AND AIRPORT RUNWAYS

By Ira B. Mullis[1]

With Discussion by Messrs. Jacob Feld; W. H. Campen,
L. G. Erickson, and J. R. Smith; and Ira B. Mullis

SYNOPSIS

This paper is intended as a guide to the rational design and construction of roadbeds for highways and airport runways. The effect of pore space, water content, and cohesion of the constituent particles of the roadbed govern its strength. Other properties are secondary ones.

INTRODUCTION

Observations and practice in highway work seem to indicate the need for a more positive method of design and construction. This statement is based on the occurrence of numerous road failures during wet, freezing weather. The statement is strengthened by an editorial in Engineering News-Record (Feb. 10, 1949) which says that the Highway Research Board meeting (EN-R, Dec. 30, 1948) issued a caution against putting too much faith in laboratory tests of soils for roadbuilding. The editorial also states that Robert R. Phillipe had said at a then recent meeting of the ASCE in New York, N. Y. that after 15 yr of soil-compaction studies there is still only a chance relationship between laboratory test results and actual performance in an embankment. As little change has been made in compaction methods since that time, it is assumed that the chance relationship continues.

Additional support to conditions cited seems to be given by Gerald A. Leonards, A. M. ASCE(1)[2] who states that in foundation and earthwork engi-

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Note.—Published essentially as printed here, in March, 1960, in the Journal of the Highway Division, as Proceedings Paper 2412. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Cons. Engr., (retired), Raleigh, N. C.

[2] Numerals in parentheses refer to corresponding items in the Appendix.

neering—despite the heralded advances of soil mechanics—experience remains the most important requisite for success.

During this research a method has been formulated that shows that when a certain laboratory method is duplicated in a roadbed, it gives the same results as that in the laboratory.

Although the fundamental principles of the method are not new, they have not heretofore been applied purposely to the design or construction of roadbeds. There are, however, numerous examples of what these principles have accomplished in all stable formations of the earth and in certain man-made embankments.

PURPOSE

The purpose of this research is not merely to discover facts about roadbeds but to reveal principles that explain the facts. Of course, when the principles are known, empiricisms are abandoned and rational methods of design and construction follow. Once the basic principles are known, the concept of the ideal becomes clear.

The ideal roadbed is one that does not become plastic in wet weather or dusty in dry weather, but remains firm and smooth without undue maintenance.

METHOD

The method of obtaining ideal roadbeds as described in this paper is the one followed by Nature herself when converting sediments into consolidated sedimentary rocks. The principles are simple and do not vary. Of course, rock types do vary in character, but the principles of strength remain constant.

As roadbeds may well be regarded as synthetic rocks, effort is made to follow the principles that Nature applies to converting gravel, sand, silt, and clay into consolidated or indurated rocks.

To spare the readers being led through the long list of erroneous hypotheses set up in an effort to discover facts about roadbeds, the results will be presented deductively and the outstanding findings set up as premises.

Premise 1.—The ability of atoms and molecules to resist force is measured by their arrangement or packing and cohesion.

Premise 2.—The ability of rock or fragments of rock to resist force is measured by the restriction of pore space and the strength of its cementing material.

Premise 3.—The weakening effect of pore space and its water content is measured by their respective volumes.

Premise 4.—Were it possible to select two hydrous materials having identical molecular arrangement and cohesion, both would break under identical loads, provided each had identical pore space and water content. This being true, the material having the lesser pore space or water content would be the stronger.

Premise 5.—It may well be that a specimen of rock or earth contains a submicroscopic structure (Premise 1) too minute to be included in the measurable pore space. In such specimens, strength may be affected more by submicroscopic structure than by small differences in pore space or water content. As in other materials of construction, one may safely resort to the method of multiple tests, statistical averages, and factors of safety as a means for the selection of material with adequate strength.

Premise 6.—The durability of rock or earth is increased by the reduction of pore space and the restriction against water losses.

Premise 7.—Cohesive earth with water-filled pore spaces cannot slake; but when its pore spaces contain considerable volumes of air, slaking proceeds as water displaces the air content.

Premise 8.—Poorly graded earth with little or no bonding material cannot be adequately consolidated by pressure or tamping alone; but it can be consolidated after suitable gradation and bonding materials have been mixed into it and sufficient water is added to cause the particles to cohere under adequate pressure or tamping.

Premise 9.—The presence of excess air in pore space provides a source of weakness. When water does become available, it is almost certain to drive out at least a part of the air, cause swelling, and weaken the roadbed in proportion to the amount of water absorbed.

Premise 10.—The effect of water on volume change and loss of strength can be alleviated or overcome by the control of water content. This may be accomplished by the reduction of pore space to a very small value and by restricting the air content to not more than 10% of the volume of the total pore space. Where air-filled pore space is very much in excess of 10% of the total, excessive water content is likely to follow with excessive volume change and loss of strength.

Premise 11.—As clay at the surface of a roadbed offers little resistance to traffic or weather, it must be protected by a more resistant material, such as a mixture of fine aggregate and asphalt. As a major purpose of the paper is to provide a basis for the establishment of a strong roadbed, the protective mat may be only of a thickness that will prevent ravelling and evaporation.

Premise 12.—If water occupies not more than 90% of the total pore space in earthwork and freezes, ample room is available for expansion.

Premise 13.—Total pore space should be about 90% saturated to avoid expansion and contraction; water losses can be held practically constant by the prevention of evaporation.

Premise 14.—When pore space is restricted to low values, the probability of 100% saturation is negligible.

Premise 15.—When total pore space is reduced to low values and 90% saturated, its resistance to load and weather will be the maximum for the given pore space.

Premise 16.—If saturation is to remain constant, the only method by which strength can be increased is by the reduction of total pore space and increased strength of the cementing substance.

Premise 17.—Every capillary system consists of three essentials—a source of water, a capillary medium such as a volume of earth of limited height that may be called a wick, and a means for the removal of water from the wick by evaporation or drainage.

Premise 18.—The quantity of water a capillary system may lift is limited by five conditions as follows: (1) quantity of water at the intake of the wick; (2) height of the wick; (3) cross section of the wick; (4) porosity of the wick; and (5) rate at which water is removed from the upper end of the wick.

Premise 19.—If any one of the limiting conditions specified in Premise 18 fails to function, capillary movement must cease at once.

Premise 20.—The conditions required for heaving or frost action are as follows: an opening of sufficient size to receive water and a freezing temperature. Openings may be in the form of a crack, joint, or even pores that may be more than 90% filled with water. Heaving will be intensified when the temperature is alternately above and below the freezing point, provided additional water is added between freezings.

Premise 21.—The least vulnerable pores are those so minute that their air content remains so firmly held that little or no water [illegible] enter. Hence it seems that formations with very limited pore space are least affected by frost action.

The method selected for obtaining the ideal roadbed is intended to provide roads that are stable under given loads in any kind of weather. The term "stability" is expressed in pounds at the moisture content of the material at the time of the test. As pore space is apt to become about 90% saturated, the utmost effort should be made to eliminate all air possible, restricting it to not more than 10% of the total pore space in the mass. Because of its simplicity and widespread use, the use of the Hubbard-Field stability apparatus is suggested.

To obtain high stability, of course, pore space must be low and bond strength high at all times. Obviously, if total pore space and air space are adequately restricted at the time of construction and the roadbed is protected against evaporation (as required in Premise 11), stability will remain very near the established value.

By this method, the stability of the roadbed is set at the value required by the weight of the traffic and the pore space and water content are restricted to produce and maintain that value regardless of weather. Rocks of constant strength are those having a constant water content and, of course, they are protected against evaporation. Moreover, these rocks do not weather as long as they remain protected from change. It seems also that roadbeds can be kept stable only by protection from change. Premise 6, based on observations of the works of Nature herself, appears not to vary.

As roadbeds are made up of rocks and sediments of many kinds, to attain maximum success one must know the basic principles of each and avoid violating them. Although one might very well begin at any point in the rock cycle, it seems best to start with igneous rocks as these were the first ones, and to continue with sedimentary and metamorphic rocks and follow with sediments. The next constructive step in the cycle is, of course, the consolidation of sediments into more stable bodies. Thus a roadbed may well be considered as moderately consolidated sedimentary rock, the strength which increases with decreasing pore space (Premise 2).

THE CRUST OF THE EARTH

The crust of the earth consists of bedrock and mantle. Of these materials, all roads and many other works of engineering are formed and, indeed, life itself depends upon them. Soil in which plants grow is formed directly from mantle or regolith and the source of mantle is, of course, bedrock.

To describe the crust of the earth in detail is not essential, therefore, the discussion will be confined to essentials only.

BEDROCK

Bedrock is classified according to origin such as igneous, sedimentary, and metamorphic rocks.(2) Sediments are referenced to William E. Twenhofel,(3,4) a recognized authority on sediments; and soil is referenced to the United States Dept. of Agriculture.(5)

Certain of these rocks, being rich in quartz and poor in metals such as iron, magnesium, calcium, and sodium, are more resistant to weathering. In this category belong granite, rhyolite, silicious sandstone, quartzite, gneiss, and others. These are known as acidic or silicious rocks which produce powders and clays of low bond strength. On the other hand, rocks poor in quartz but rich in iron, magnesium, calcium, and sodium are less resistant to weathering but are tougher and, with certain exceptions, produce rock powders and clays of higher cementing value.

Pore Space in Rocks.—Tables by Heinrich Ries and T. L. Watson(6) show that the pore space in many building stones is often less than 1.0% and if one excepts sandstones, limestones and vesicular rocks, all have a total pore space of less than 2.0%. The source mentioned states that sandstones and limestones have a wide range. Sandstones are listed as having a porosity ranging from 6.9% to 25.5% and limestones 0.32% to 13.38%. Vesicular rocks may have a porosity of more than 25%. Of course, rocks with high pore space, weather more rapidly than do the same rocks with less pore space.

After rock has been crushed, the pore space between fragments and particles is high. Porosity here may be 40% or more.

Porosity-Strength Relationship.—Although the effect of pore space on the strength of rocks is well known, the relationship seems to be almost entirely overlooked in practice. The relationship is clearly shown in Table 1 in which granites are found to weaken with increasing saturation. A similar relationship between porosity and strength is also found in other rocks. Indeed it seems to be almost, if not entirely, a universal relationship in brick, concrete, and timbers (Premise 2). This fact is also sometimes stated that "every aggregation of matter attracts every other aggregation of matter by a force which is directly proportional to their respective masses and inversely proportional to the square of the distance between them", that is, across the pore spaces. This law is valid between atoms where space is measured in Angstrom units and also in the celestial sphere where distances are measured in light years. Thus, if the unit of distance is 1, the attraction is 4 times that where the distance is 2.

Another example is given by Edward J. Dana(7) showing the effect of molecular arrangement (packing) on hardness, which is a manifestation of strength. Diamond and graphite are both carbon, but the diamond is the hardest of all minerals and graphite one of the softest. Difference is attributed to that of specific gravity, the diamond being much the greater (Premise 1).

Effect of Water on Strength of Rocks.—The weakening and softening effect of water on minerals and rock is a well-established fact. For example, gibbsite and diaspore consist entirely of alumina and water, but gibbsite is the softer as it contains 34.6% of chemically combined water and diaspore only 15.0%. The hardness of gibbsite is 2.5 to 3.5 but diaspore is 6.5 to 7 each according to the Mohs' scale.(7) Of course it is well known that hydrous structural materials such as stone, brick and concrete are softer when wet.

Magnitude of Saturation of Rocks.—D. W. Kessler and W. H. Sligh(8) found that the absorption of limestones is proportional to total pore space, but when it is thoroughly dry, a long period of soaking is necessary to completely refill the pores. Two-week immersion tests showed the average saturation to be 70% and that specimens soaked for 6 months were less than 90% filled. Table 2 shows the percentage of absorption by pore volume under three methods and only those under vacuum became more than 85% filled.

TABLE 1.—EFFECT OF PORE SPACE AND WATER CONTENT ON STRENGTH OF GRANITES[a]

Items	Group 1		Group 2		Group 3	
	Saturation capacity 0.00%-0.10%		Saturation capacity 0.11%-0.20%		Saturation capacity 0.21%-0.30%	
	Tested		Tested		Tested	
	Dry	Wet	Dry	Wet	Dry	Wet
Maximum strength, in psi	53,800	41,500	44,100	40,000	33,200	33,000
Minimum strength, in psi	13,800	14,400	15,600	12,000	13,800	13,000
Average strength, in psi	31,200	26,200	25,500	23,100	23,900	21,700
Percentage dry strength, based on dry condition of Group 1	100	. . .	82	. . .	76	. . .
Percentage wet strength, based on dry condition of each group	100	84	100	91	100	91

[a] Data from Kessler, Daniel W., Herbert Insley and William H. Sligh, Physical Mineralogical and Durability Studies on Building and Monumental Granities of the United States. National Br. Stds., Research Paper RP 1320, vol. 25, August 1940.

Notes.—Saturation was measured after 48 hr submergence in water; but it was found that saturation was not complete, even after 1 yr. Indeed the greater portion of the specimens were less than one-half filled at the end of the year.

The term "pore space" as used in the title of the table represents "saturation capacity," that is, the water absorbed during the 48-hr period of inundation in water. Although these terms are known to be not entirely accurate, they seem to be the most appropriate terms that have so far occurred.

Bureau of Public Roads, Results of Physical Tests of Road-Building Aggregate shows the average absorption of granites as 0.3%, with 80% of less than 0.4%. Less than 10% of the granites had an absorption of 0.1%.

Many absorption tests made by the Bureau of Public Roads(8) show the resistance of many rocks to absorption. As one would expect, the finer grained or least porous rocks absorb the least while sandstones and limestones absorb the greatest amounts.

Effect of Weathering on Rocks.—The weakening effect of weathering has a marked effect on the strength of rocks. Granites are considerably weakened when one can first detect the loss of luster from weathering. For example, experiments made on granite as a part of this research, showed that the slightest loss of luster that could be detected by the eye was attended by a loss of 1% as measured by the rattler test. Again, a granite weathered to the extent miners refer to as "sap", an early stage, showed an abrasion loss almost two times that of fresh rock in the same quarry.

Rocks that weather most rapidly are the basic cellular ones that are readily attacked by the agencies of weathering. Of course, these agencies are of an acidic nature. Rocks least vulnerable to weathering are acidic rocks, such as granite and rhyolite which are of an acidic nature.

SEDIMENTS

Definition.—Sediments, another name for the loose materials of the earth's crust, are defined by Twenhofel(10,11) as deposits of solid material (or material in transportation which may be deposited) made from any medium on the earth's surface, or in its outer crust under conditions of temperature approximating those normal to the surface. Some transportation is involved, but it may range from mere detachment of the particles to the parent rock to movement over many miles; it must, however, be sufficiently great to destroy textures and structure of the parent rock, except as these are preserved in the particles.

TABLE 2.—PERCENTAGE ABSORPTION BY PORE VOLUME[a]

Material	Absorption after rapid submersion	Absorption after slow submersion	Absorption under vacuum
Sandstone	52.97	61.30	85.46
Sandstone	61.08	64.88	95.48
Marble	59.17	84.27	94.67
Limestone	35.46	37.20	90.04
Slate	72.92	79.16	100.00
Tuff	65.51	69.37	89.64
Granite	41.20	57.71	85.54

[a] Ries, Heinrich & T.L. Watson, Engineering Geology, p. 437, 1914.

Source of Sediments.—Sediments, as stated in the definition, must come from the parent rock. Thus it can be seen that sediment may be separated from the parent rock by a mere fraction of an inch or it may be measured in many miles. It may also be transported by water, air, ice, or animate life. It may be derived from any type of rock, depending on source and transporting agency.

Sediments may be derived from organic matter also. Certain areas of the Earth abound in sediments such as microscopic tests, shells, peat, etc.

Classification According to Size.—As reported by Twenhofel,(10,11) sediments are classified from boulders to clay size, giving the limiting dimensions of the pieces, the aggregate name, and the name of the indurated rock formed of the pieces. Descriptions are given of the several sediments.

Pore Space in Sediments.—Pore space in sands with fine particles screened out seems to average about 40%. Table 3 shows that the usual deviation from 40% is usually not more than about 5%. Using the maximum and minimum values for spheres as computed by Slichter (Table 3), the average value 36.70% (item 3) is close to the average experimental value by H. J. Fraser and also that by A. E. R. Westman and H. R. Hugill, each using lead shot in the experiment (items 4 and 5). Values of washed sands and other aggregates shown in Table 3 (items 9 and 10) by Fraser, indicate the effect of water in the bulking of wet materials. In the work cited he shows also, that dry packing of sands, lead shot, etc., is more efficient than wet packing. Clays, however, require a limited amount of water for packing.

TABLE 3.—PORE SPACE IN CERTAIN MATERIALS

Item	Material	Percentage
1	Slichter Computed pore space between spheres - max.,	47.64
2	Slichter Computed pore space between spheres - min.,	25.95
3	Slichter Average of 1 and 2,	36.79
4	Fraser Actual pore space in lead shot - avg. dry compact.	37.18
5	Westman & Hugill Compacted lead shot - avg. of 4,	36.9
6	Fraser Std., beach, and dune sands - avg. dry compact,	36.31
7	Westman & Hugill Rounded Sands, Nos. 4 to 65 - avg. of 3,	38.2
8	Westman & Hugill Rounded sand, Nos. 150 to 200 - 1 only,	42.5
9	Fraser Beach, dune and standard sands, dry loose - avg. of 3,	40.29
10	Fraser Beach, dune and standard sands, wet loose - avg. of 3,	44.80
11	Mullis Washed, rodded and dry concrete sands - avg. of many,	39.9
12	Mullis Washed, rodded and dry concrete sands - max. of many,	41.0
13	Mullis Washed, rodded and dry concrete sands - min. of many,	36.3
14	Mullis Washed, crushed rock aggregates - avg. of 125,	39.1
15	Mullis Washed, crushed rock aggregates - max. of 125,	41.3
16	Mullis Washed, crushed rock aggregates - min. of 125,	26.8
17	King, reported by Meinzer, Clays - avg. of many,	35
18	King, reported by Meinzer, Clays - max. of many,	47
19	King, reported by Meinzer, Clays - min. of many,	26
20	Meinzer Glacial outwash, silt & clay, Connecticut,	18.0
21	Meinzer Till or boulder clay, Connecticut,	13.5
22	Meinzer Till or boulder clay, Connecticut,	12
23	Meinzer Till or boulder clay, Connecticut,	11.5
24	Meinzer Lake deposit, silt, Connecticut,	36.0
25	U.S. Dept. Agr., reported by Meinzer, Soils, Avg. of many,	55
26	U.S. Dept. Agr., reported by Meinzer, Soils, max. of many,	65
27	U.S. Dept. Agr., reported by Meinzer, Soils, min. of many,	45

References, by items:

1,2 Meinzer, O.E., The Occurrence of Ground Water in the United States, U.S.G.S., Water Supply Paper 489, 1923, pp. 4-5.
4,6,9,10 Fraser, H.J., Experimental Study of the Porosity and Permeability of Clastic Sediments, J. Geol., v. XLIII, No. 8, 1935, p. 936.
5,7,8 Westman, A.E.R. & H.R. Hugill, The Packing of Particles, J.Am.Cer.Soc., v. 13, No. 10, Oct. 1930, pp. 767-779.
11-16 Mullis, Ira B., Private record, unpublished.
17-19 King, F.H., reported by Meinzer, Op. cit., p. 10.
20-24 Meinzer, O.E., Op. cit. p. 11.
25-27 U.S. Dept. Agr., reported by Meinzer, Op. cit. p. 10.

The range of pore space in fine-grained sediments found in natural deposits is large, ranging up to 90% by volume in recently deposited muds.(12) The porosity of clays and shales, deposited probably in water, and buried at great depths is discussed by Hollis D. Hedberg.(13) A detailed discussion of the porosity of rocks with many data on pore space is given by Oscar E. Meinzer.(14) Many other data are given in Table 3.

Effect of Pore Space and Water on Strength.—Where pore space or water content is large in sediments, there is little strength. Deposits of silt (loess) in northeastern Iowa, locally known as "soft spots", are found at shallow depths beneath the ground surface with about 37% pore space (Table 4, No. 14). The pore space is practically filled with water at all times. The water content being adequate to produce a very plastic condition, the ability of the silt to support traffic is inadequate, even for light traffic, unless the bed of silt is deeply buried.

TABLE 4.—EFFECT OF POROSITY ON THE STRENGTH OF CERTAIN MATERIALS

Item	Type of Material	Porosity, in %	Strength (wet) lb/in^2
1	Siltstone, footing Kans. R. Bridge, Topeka, Kans.	5.0	High
2	Shale, well section, Venezuela	9.07	5874
3	Shale, English coal measure, average	13.2	High
4	Gravel-loess, Omaha Airport after 1943 flood	14.6	Good
5	Shale, well section, Venezuela	16.8	4420
6	Shale, well section, Venezuela	25.32	2595
7	Clay, average of 19 American clays	26.38	?
8	Shale, well section Venezuela	31.33	820
9	Roadbed, Omaha Airport, silty clay	31.9	200
10	Roadbed, Omaha Airport, silty clay	32.5	100
11	Roadbed, Omaha Airport, silty clay	32.6	Fair
12	Shale, well section Venezuela	33.63	260
13	Roadbed, Omaha Airport, silty clay	34.1	55
14	Roadbed, typical loess "soft spot," N.E. Iowa	37.0	Very low
15	Roadbed, Omaha Airport, silty clay	41.0	13
16	Roadbed, Omaha Airport after '43 flood, avg.	32.6 Saturated	94%
17	Roadbed, do. max.,	35.5 Saturated	94%
18	Roadbed, do. min.,	30.5 Saturated	90%
19	Pavement base, do. avg.,	14.6 Saturated	78%
20	Pavement base, do. max.,	17.8 Saturated	75%
21	Pavement base, do. min.,	12.5 Saturated	78%

References, by items:

1 Mullis, Ira B., Unpublished test, only 2 percent moisture content. Like other wet earth, it slakes only when dry.

2,5,6,8,12 Hedberg, Hollis D., The Effect of Gravitational Compaction on the Structure of Sedimentary Rocks, Bull. Am. Assn. Petrol. Geologists, v. 10, pt. 2, pp. 1035-1072.

3,7 Leith, C.K. and W.J. Mead, Metamorphic Geology, pp. 69-72, 1915.

4,9,10,11,13,15 Campen, W.H., Discussion, Principles Applying to Highway Roadbeds, by Ira B. Mullis, Trans., ASCE, v. 104 (1939) pp. 1403-1405.

16-21 Campen, W.H. and J.R. Smith, The Behavior of Densified Soil-Water Mixtures Under Very Adverse Conditions, Proc. Highway Res. Bd., v. 23, pp. 450-459, Tables 4 and 6.

14 Mullis, Ira B., Unpublished report.

In contrast to the Iowa silt, another silt far below the bed of the Kansas River at Topeka (Table 4, No. 1), supports a reinforced concrete bridge. The latter bed of silt is so hard that excavation was made only by the use of drills and explosives. The water content was 2% and the pore space about 5%. This silt gives adequate support to the bridge.

Loss of bearing capacity of sediments as in solid rocks seems to be a condition due to increase in pore space and decrease in bond between particles. Hence, if pore space is small and bond is strong, bearing capacity also will be large; however, any increase in pore space or weakening of the bond lessens the strength of the mass. Also, in a given pore space, a weak bond is destroyed by a small increase in water; but a strong bond is destroyed only by a correspondingly large increase in water. Hence, the strength of coherent earth is measured by pore space and strength of bond.

Slaking.—Slaking seems to occur only when dry or partly dry coherent earth is in contact with water. All seem to know that dry earth begins to disintegrate or slake in water, but no record has been found stating that sediments with their pore spaces filled with water do not slake. Observation of this phenomenon first came within the experience of this research during the construction of Utah FAP 51-A on U.S. 40, between Salt Lake City and the Utah-Nevada boundary line in 1923. Moist blocks of the compacted silty clay that had fallen into salty water had not slaked after more than a year of submergence. Specimens of other types of earth molded into a moist compact condition, failed to slake in fresh water after months of submergence. Certainly wet earth can be eroded, though it does not slake.

During excavations in a bed of consolidated silt, many feet beneath the bed of the Kansas River, the material was so hard that removal was done only by means of drills and explosives (Table 4, No. 1). Chunks of the material did not slake in water as pore space was already filled; but on drying to the point where hair cracking began, it slaked rapidly to a cream-like consistency.

Properties of Silt.—Silt produced in high latitudes is mainly the result of rock grinding, such as that done by glaciers, and consists primarily of fresh rock flour with little decomposition; consequently, its bond strength is negligible. In contrast, silt produced in the tropics is apt to undergo more decomposition with disintegration; therefore, its bond strength tends to be somewhat greater than that of fresh silt. The properties of tropical silt, therefore, may be considered as intermediate between polar silt and clay.

Beds of silt buried at depths that prevent evaporation, tend toward a constant water content. This condition has been observed many times in thick beds of loess where the pore space may be only partly filled; however, certain beds of loess in northeastern Iowa with 1 ft or more of covering is invariably almost saturated (Table 4, No. 14). Beds of silt with little or no clay content seem to tend toward complete saturation under favorable conditions, though more consolidated beds of silt do not usually exceed 90% saturation. Many similar observations have been made on the moisture content of silts, one set of which is included in Table 4, No. 16 to No. 21. The percentages of saturation of the roadbed and pavement bases after they had been covered in water during a period of 10 to 17 days were obtained after the flood had subsided. That a limited pore space seems to restrict the percentage of saturation is further supported by Table 1 which shows that the strongest granites absorb the least water. Table 2, where the coarser-grained rocks also absorb the least water is shown in the work of Kessler and Sligh.(8)

Owing to their high water content, silts in freezing temperatures are prone to produce frost boils and heaving. This characteristic will be discussed under the heading "Frost Action".

Properties of Clays.—Clays are aptly described by T. A. Klinefelter and H. P. Hamlin(15) as being quite literally geological trash, materials resulting from the decomposition of rocks and minerals. They are mixtures of all sorts of debris, including not only the clay minerals themselves, but a variety of impurities. Their most prominent property to the potter or brickmaker is that of plasticity, but to the roadbuilder it is that of stickiness when wet and its ability to maintain a bond between particles with which it is in contact.

Classification of Clays.—Although a number of classifications have been proposed, only three are utilized herein. They are as follows:

1. Chemical composition and environment of the parent material;
2. Strength of the clay; and
3. Usefulness of the clay in roadbuilding.

The effect of chemical composition pointed out in the discussion of igneous rocks, shows that powder of basic rocks have greater cementing strength than does powder of acidic rocks. It has also been shown by Allerton S. Cushman(16) that actual decomposition of rock powder enhances the binding power of these powders. He also shows the profound effect of wet-grinding of rock powder on cementation.

Strength of Clays.—Although the use of clay as a binder in crushed rock pavements is usually regarded as poor practice, work by Logan W. Page and Cushman(17) found that certain clays have much greater bond strength (cementing value) than did any rock tested. For example, a calcareous clay from Florida had a cementing value in excess of 4,000; four clays from Virginia ranged from 1,500 to 3,059. Also the average cementing value of a number of clays was in excess of 1,200 and that of shales more than 400. In contrast, the Bureau of Public Roads(18) considers a cementing value below 25 as low, that between 26 and 75 as average and that above 75 as high. It might well be stated that only certain basic rocks, usually altered ones, test as high as more than 500, the highest value recorded, in the bulletin. As values in excess of 500 are so unusually large, values in excess of 500 are recorded as 500+. Limestones and dolomite have been known to test as 100 or higher.

The probable reason for regarding the use of clay as a binder for crushed rock pavements as poor practice, may well be attributed to the past lack of adequate facilities for drying, pulverizing, and proportioning the binder and the aggregate. Topsoil once was not favored as a binder, but now this is considered good practice. Observation of a case of the successful use of a clay as a binder for a crushed rock road has come within the observation of this research. Although machinery for the preparation and mixing of binder and crushed rock was not available, a satisfactory surface was maintained for slow-moving traffic some 10 or 15 yr. It would seem that with the laboratory and construction facilities now available, the development of a satisfactory clay-aggregate base is urgent. The use of topsoil should be discouraged because of the many years required to develop a soil that will grow crops.

Experience with sand, gravel, and crushed rock laid in thin layers on gumbo and other heavy types of compacted clay can be made to become smooth, stable, and stronger than the less sticky clays. Of course, like all clay-bonded aggregates, the surface must be treated with a sand-asphalt covering to protect it against ravelling during dry weather.

For the convenience of those who may need preliminary information on the strength of certain types of clays as binders, Tables 5, 6, 7 and 8 are presented. As may be seen, the authors of the tables were recognized leaders at the time they were written. The tables are intended to give the research worker preliminary information to guide him in the selection of binders. Final selection should be made only after additional tests of available clays. His selection may then be compared with clays in the tables for comparison, after which he is able to judge the strength of the clay selected. Additional information on clays may be found in reports on clays issued by most of the states.

Effect of Parent Material and Drainage on Clay Type.—Clays weathered from acidic rocks such as granite and rhyolite tend, except under conditions described by Clarence S. Ross and Sterling B. Hendricks,(19) to be of the kaolin type. This type is one of the weakest and most fragile clays when dry, and has little stickiness when wet. This type of clay is usually, if not always, developed on slopes where drainage is good. However, where drainage is poor and alkalies or alkaline earth is present, granite may weather into tough sticky clay.(19)

TABLE 5.—MODULUS OF RUPTURE OF CLAYS, IN POUNDS PER SQUARE INCH

Description	Modulus
Low	0 to 100
Medium low	100 to 200
Medium	200 to 400
Medium high	400 to 800
High	Over 800

[a] Watts, A.S., Am. Ceram. Soc., VI, p. 247, 1920.

TABLE 6.—MODULUS OF RUPTURE OF CERTAIN CLAY TYPES, IN POUNDS PER SQUARE INCH[a]

Type of Clay	Modulus
Washed kaolin	75-200
White sedimentary kaolin, Ga., S.C.	150-166
Ball clay	25-600
Crucible Clay	187-691
Refractory bond clay	395-1093
Glass pot clay	173-1068
Sagger clay	46-474
Stoneware clay	94-678
Sewer pipe clay	190-589
Brick clay	50-1500

[a] Ries, Heinrich, Clays, Their Occurrence, Properties and Uses, 3rd Ed., p. 212, 1927.

TABLE 7.—COMPARATIVE STRENGTHS, IN POUNDS PER SQUARE INCH, OF CLAYS WITH AND WITHOUT SAND[a]

Specimen	Without sand			With 50% sand		
	Compression	Tensile	Transverse	Compression	Tensile	Transverse
Ball clays	565-1148	135-210	375-558	464-777	124-180	242-330
Plastic clays	631- 954	155-172	484-520	476-553	113-150	216-280
Shales	636- 806	126-187	311-403	403-449	77-111	178-237
Plastic kaolins	455- 539	104-147	239-325	286-559	54-110	122-210
Primary kaolins	205- 349	34- 69	74-166	164-172	29- 35	56- 82

[a] Ries, Heinrich, Clays, Their Occurrence, Properties and Uses, 3rd Ed., p. 221, 1927.

Clays weathered from basic rocks such as basalt and gabbro tend, except under unfavorable conditions described by Ross and Hendricks, to weather into a montmorillonite type of clay. This type is very sticky when wet and tough when dry. Bentonite and gumbo clays of the Mississippi Valley belong to the montmorillonite type. Iredell, Susquehanna, and White Store clays of the southeastern part of the United States(19) also belong to the montmorillonite type. Strength of several types of clay, including kaolin and gumbo, are shown in Table 8. Tables 5, 6, and 7 also show relative strengths of a number of clays.

Clays derived from the weathering of limestone, also a basic rock, when found on poorly drained areas with abundant rainfall are noted for stickiness when wet and toughness when dry. Houston and Sumter clays developed from soft chalky limestones in the warm humid climates of Texas and Alabama, respectively, are extremely sticky when wet and tough when dry. Again, it should be stated that these are developed on flat lands.(20) In contrast, Brackett and Clark sediments, also developed from well-drained calcareous materials under arid conditions on the plains of Texas and Kansas, respectively, contain little clay and hence little stickiness.(21) Thus one may again see the effect of temperature, rainfall and drainage on the type of sediment derived from rocks.

Research on Clays.—In recent years several organizations have contributed much knowledge useful to the highway engineer. Among these the United States Geological Survey and the American Ceramic Society are important contributors. Excellent work and an extensive bibliography have been published by Ross and Paul F. Kerr,(22) another by Ross and Hendricks and a book by Ralph E. Grim.(23) The textbook on soils by L. D. Baver is also suggested.(24)

TABLE 8.—TENSILE STRENGTHS IN POUNDS PER SQUARE INCH, OF 135 MISSOURI CLAYS[a]

Clay	Range	Average
Flint-clays	8 to 50	20
Kaolins	12 to 20	20
Fire-clays and pottery clays	50 to 284	150
Shales	87 to 192	120
Gumbo	275 to 410	340
Loess	97 to 354	150

[a] Mo. Geol. Survey, XI, p. 111. Recorded by Reis, Heinrich, Clays, Their Occurrence, Properties and Uses, 2nd Ed., p. 150, 1914.

SOILS

Definition.—A short specific definition of the word "soil" is one given in a glossary of the United States Department of Agriculture(25) as "The natural medium for the growth of land plants". Simonson has described(20) "What Soils Are" and Russell(20) described their "Physical Properties". Chester R. Longwell and Richard F. Flint(26) also add to the foregoing information on soils.

Moreover, every soil must consist of mineral and organic matter, water and air; it must have a profile of two or more layers nearly parallel to the land surface; and it must also contain bacteria and fungi to break down the organic matter for the use of plants.

In contrast, materials best adapted to the construction of roadbeds are those devoid of organic matter and should have water and air contents restricted to very small percentages. However, as grasses and plants are essential to the prevention of erosion and to the preservation of natural beauty, soil should be stripped prior to grading operations for the support of grasses and plants along the roadway.

As organic matter should not be included in roadbeds and is essential as a medium for roadside development, it should be kept apart from infertile sediment and each utilized in its place. Both types cannot, with impunity, be called "soil." It leads to confusion. For example, during a field check of the road plans, the horticulturist on roadside development specifies that fertile "topsoil" in a given area shall be stockpiled for plants and in another area the "soils engineer" specifies infertile "topsoil" for the improvement of the top of the roadbed. Each refers to the respective materials as "topsoil." Obviously, much confusion arises. Although the word soil, through priority of use, should be restricted to its agricultural meaning, the habit of applying it to all sediments or mantle is so firmly fixed in the highway field that one can hardly hope for change.

Source of Soil.—The source of soil is, in part, from parent material (sediment), another part is from the decay of plant and other organic matter and still other parts from water, air, etc.

Rate of Soil Formation.—As soil scientists agree that the rate of soil development is so slow and its development requires so much longer time than a human generation, it must be conserved. Some guess that a fair soil might develop in 100 or 200 yr,(27) and the shortest so far found for a volcanic ash to develop into soil in a hot humid climate is 45 yr.(27) It is, therefore, apparent that as infertile sediments are more valuable for making roadbeds, soils should be conserved for the growing of plants for food and other purposes for which infertile sediments are not adapted.

Contrasts Between Productive Soils and Stable Sediments.—Although the difference between productive soils and stable sediments seem apparent at a glance, one often hears the word "soil" applied to any loose earthy material properly called mantle or sediment. The basis of this statement is not the mere opinion of the writer, but it rests upon well-established practice and authentic treatises on soils written long before any organized work had been started on materials for roadbeds. For this reason, agricultural workers have the right of priority of use of the word soil. Evidence of this fact is cited by Twenhofel(10) who states that the field of geology now known as sedimentation had its beginning during the last quarter of the nineteenth century and that "The first decade of the present century (1939) witnessed the slow rise of sedimentation as a division of geology". As everyone knows, science demands the use of the most precise terms possible to avoid misunderstanding. As there are many kinds of soil and sediment, it seems that for the sake of clarity one should use a more descriptive word unless he is speaking in the most general terms. It also seems that more emphasis should be given to type of sediment and its state of consolidation.

For these reasons the following contrasts are listed:

Productive soils	Compact sediments
1. Do not support heavy loads	1. Support heavy loads
2. Contain much pore space	2. Contain little pore space
3. Have large capacity for water	3. Have small capacity for water
4. Possess high permeability	4. Possess low permeability
5. Require abundant air content	5. Require little air content

6. Require much organic matter	6. Require no organic matter
7. Need abundant bacteria	7. Need no bacteria
8. Are well-suited for plants	8. Are ill-suited for plants

Productive soils on slopes need surface drainage, but those on flats need both surface and sub-drainage. Of course, compacted areas such as barnyards need no sub-drainage, though they may need surface drainage to prevent erosion.

ROADBEDS

Definition.—A roadbed is defined by the Merriam-Webster Dictionary as follows: "In railroads, the bed on which the superstructure (ties, rails, etc.) rests; also, the ballast on which the ties rest. In common roads, the whole material laid in place and ready for travel".

In this paper, however, the roadbed is restricted to the body of compacted earthy matter, including the protective surface or pavement base. Laterally, it extends from edge to edge of the roadway and in depth it extends to firm ground or to ground that is confined to the extent that it cannot flow under the anticipated load.

As the construction of the roadbed approaches the elevation of the subgrade, that is, the protective surface, pore space should be reduced to a low value and the bonding clay utilized should be of excellent quality. It is, of course, in the upper part of the roadbed where the greatest strength is needed and considerable effort should be exercised to develop the utmost strength attainable from local materials. Obviously, if adequate strength is attained throughout the roadbed, the necessity of a thick and expensive pavement can be lessened and perhaps avoided. If, perchance, the roadbed is composed of a coarse well-graded sand mixed with an adequate amount of strong clay binder, compacted until the total pore space is not more than about 15% and shows a high value of stability, it might well serve as a pavement base also.

Strength of Roadbed Materials.—Before designing the roadbed it is imperative that the strength of local materials, that is, materials of which the roadbed is to be formed, be determined. As moist earth is the material that must be utilized to the limit of its capacity for strength, it should be pre-tested to determine its stability when its pore space is reduced to the lowest practical value and about 90% filled with water. A convenient apparatus for making this test is the well-known Hubbard-Field instrument designed for use in measuring the stability of asphalt mixes.(28)

Observations cited in the Introduction seem to indicate that the empirical method of constructing roadbeds by compacting them to "maximum density at optimum moisture" is not adequate. This should be expected as the predicted strength is measured only by the input of energy applied to compaction. Of course, many examples have been cited showing that the method is adequate for certain well-graded materials, but this might well be regarded as coincidence since output of energy is rarely equal to input. Sand of a uniform size, like ball bearings, cannot have its pore space reduced except through the breakage of the units of matter. Although strength of materials may be estimated by various methods, accuracy is never attained except by weighing.

The Density-Strength Relationship.—The most prevalent method for estimating the strength of bodies of earth at the present time (1960) is that known as the density-strength relationship. As the actual material of which wood is composed has an apparent specific gravity, defined by ASTM (E 12-27), of about

1.54 for all species, it is apparent that for a given bulk specific gravity (bulk density, to be brief), pore space is almost constant. For this reason, the statement is frequently made that the density-strength relationship is a splendid one. In contrast, the apparent density of rocks and sediments has a wide range. For example, where the apparent density of rocks ranges between 2.50 and 2.96, the computed pore space may have a range of as much as 10% in sediments having a bulk density of say 1.70, 1.80, or 1.90. Where hematite or other heavy rocks abound, variation in pore space became correspondingly greater. This range in variation of bulk density for a certain value of pore space is an adequate reason for expressing condition of compaction in terms of total pore space instead of bulk density.

TABLE 9.—INTERRELATIONSHIP OF PORE SPACE, WATER CONTENT AND STRENGTH[a]

Number	Density in grams per cubic centimeter		Percentage of water, by weight			Percentage total pore space	Percentage original saturation	Strength, in pounds per square inch
	Apparent	Bulk	Capacity	Content (original)	Content (future maximum)[b]			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
6	2.72	1.99	13.5	11.5	12.2	26.8	85.1	3,000
20	2.72	2.05	12.0	11.0	10.8	24.5	91.6	2,500
23	2.72	1.97	13.9	11.4	12.5	27.5	82.0	2,000
21	2.72	1.99	13.4	12.0	12.1	26.6	89.5	1,500
19	2.72	2.05	12.0	11.2	10.8	24.5	93.4	1,450
8	2.72	1.95	14.5	12.5	13.0	28.2	86.2	1,400
1	2.72	1.94	14.7	13.0	13.2	28.6	88.5	1,200
18	2.72	1.97	13.9	12.0	12.5	27.5	86.2	1,000
5	2.72	1.76	20.0	13.5	18.0	35.3	67.5	850
14	2.72	1.84	17.6	15.2	15.8	32.3	86.2	750
10	2.72	1.63	24.7	22.5	22.2	40.0	91.0	650
7	2.72	1.75	20.4	18.5	18.4	35.6	90.0	525
2a	2.72	1.75	20.4	16.8	18.4	35.6	82.2	425
7a	2.72	1.73	21.0	18.8	18.9	36.4	89.5	300
16b	2.72	1.49	30.4	29.3	27.4	45.2	96.4	120[c]

[a] Basic data from Proctor, R.R., Fundamental Principles of Soil Compaction, Eng. News-Record, Sept. 21, 1933, p. 350. [b] Estimated as 90% of value of Col. 4. Future strength will be lessened only when Col. 6 exceeds value shown in Col. 5. [c] Rejected for use as the specifications require a strength of not less than 300 psi.

It now seems essential that the history of the method, "maximum density at optimum water content" theory be reviewed and the original purpose clarified. The paper by R. R. Proctor(29) states that the method is designed to secure the compaction of soils to the density which by actual test is essential to obtain water-tightness and stability of the dam. Water-tightness or rate of percolation and stability were restricted to definite amounts. Minimum stability was set at 300 psi when the pore space was saturated with water. Where resistance to load was less than 300 psi, it is stated that the weight of the roller must be increased in approximately the same ratio as the increase in the required needle reading. Moreover, it should be emphasized that, as noted previously, the measure of water-tightness and stability were determined by the

TABLE 10.—INTERRELATIONSHIP OF PORE SPACE, WATER CONTENT AND STRENGTH

Number	Density in grams per cubic centimeter		Percentage of water, by weight			Percentage of total pore space	Percentage of original saturation	Strength, in pounds per square inch
	Apparent	Bulk	Capacity	Content (original)	Content (future maximum[a])			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
3	2.72	1.73	21.0	16.0	18.9	36.4	76.1	57
5	2.72	1.70	22.0	16.3	19.8	37.5	74.0	55
13	2.72	1.66	23.5	17.3	21.2	39.0	73.5	48
4	2.72	1.60	25.7	16.1	23.1	41.1	62.6	47
6	2.72	1.70	22.0	16.4	19.8	37.5	74.5	47
8	2.72	1.63	24.6	16.7	22.1	40.0	67.8	41
10	2.72	1.70	22.0	17.0	19.8	37.5	77.2	41
17	2.72	1.63	24.6	17.8	22.1	40.0	72.3	36
14	2.72	1.58	26.6	17.4	23.9	41.9	65.5	36
1	2.72	1.58	26.6	15.1	23.9	41.9	56.7	34
28	2.72	1.71	21.7	19.4	19.5	37.1	89.2	33
25	2.72	1.71	21.7	19.3	19.5	37.1	88.9	32
2	2.72	1.56	27.3	15.8	24.6	42.6	57.9	30
32	2.72	1.69	22.4	20.0	20.2	37.9	89.2	30
23	2.72	1.71	21.7	19.0	19.5	37.1	87.5	28
15	2.72	1.68	22.7	17.6	20.4	38.2	77.5	28
29	2.72	1.68	22.7	19.5	20.4	38.2	85.9	27
24	2.72	1.72	21.4	19.1	19.2	36.7	89.1	26
35	2.72	1.66	23.5	20.9	21.2	39.0	89.0	25
27	2.72	1.69	22.4	19.3	20.2	37.9	86.0	23
18	2.72	1.69	22.4	18.4	20.2	37.9	82.0	23
21	2.72	1.65	23.8	18.9	21.4	39.3	79.4	22
38	2.72	1.65	23.8	21.7	21.4	39.3	91.1	22
42	2.72	1.62	24.9	22.5	22.4	40.4	90.1	18
39	2.72	1.63	24.6	21.7	22.1	40.0	88.1	17
43	2.72	1.61	25.3	22.7	22.8	40.8	89.7	16
41	2.72	1.60	25.7	22.5	23.1	41.1	87.5	14
40	2.72	1.58	26.6	22.1	23.9	41.9	83.0	13
47	2.72	1.57	26.9	25.4	24.2	42.2	91.0	13
45	2.72	1.55	27.8	24.6	25.0	43.0	88.5	11

[a] Percentage water based on dry weight of the material. Probable future water content based on a widespread experience that total pore space does not usually become more than 90% saturated. Future strength will be lessened only when Col. 6 exceeds value in Col. 5.

actual test of percolation and stability was measured by means of needle readings expressed in pounds per square inch, but not from the bulk density of the dam.

Density, it seems, was considered merely as a means for obtaining watertightness and stability. Laboratory compaction was accomplished by means of a 5-1/2-lb hammer applied with firm 12-in. strokes. Later, someone changed the firm 12-in. strokes to 12-in. drops. The change became standard procedure. The difference in results of the two methods may be found by comparing Table 10 with Table 9, the original method. Still later, in an effort to further increase the bulk density, the weight of the hammer was increased to 10 lb and the drops to 18 in. Various methods, such as lessening the thickness of layers and applying a greater number of strokes with the hammer have also been added. These changes evidently indicate a lack of entire satisfaction of results offered by previous methods.

Effect of Total Pore Space and Water Content on Strength.—Instead of measuring the energy applied to attain density, it seems more logical to measure results in terms of resistance to load and weather. The superiority of measuring quality by ability to resist weather and load will be exemplified in Tables 9, 10, and 11.

Original data in Tables 9, 10, and 11 consist of only that in Cols. 1, 2, 3, 4, and 9. The other data are computed from the original data. To facilitate computations, the original data which were expressed in pounds per cubic foot, have been converted into grams per cubic centimeter. The convenience of this system should be apparent in the tables. Reasons for arranging the data in tables in the form here used, instead of the usual density, water content, air voids and strength graph, are as follows: Cols. 2 and 3 control the value in Col. 7 and facilitate checking; Cols. 3 and 7 govern the value in Col. 4; Col. 4 indicates the capacity to contain values in Col. 5; Col. 6, computed 90% of Col. 4, indicate the probable maximum saturation to be expected; Col. 8 is governed by the relationship of Col. 5 to Col. 4 at the time of construction; and Col. 9 indicates the strength at the time of construction.

The advantage of having several tests of identical samples arranged as in Table 11 enables one to analyze the effects of certain properties on strength at the time of construction and also in the future. Such data provide opportunity to know the present strength and that of the future also. If, for example, one has information or an idea that a certain compacted sediment is not likely to attain 90% of saturation, he is in a position to soak it after a lower percentage of saturation and determine the more probable limit to be expected. Again, the need of making multiple tests is stressed.

Before leaving Table 9, comments are made as follows: of the 14 accepted specimens tested, 5 had a total saturation of 90% or greater, 6 had a value of between 85% and 89% and the remaining specimens less than 85%.

Of the 30 tests shown in Table 10, 3 have a saturation value of 90% or greater, 11 have a value between 85% and 89% and the remainder a value less than 85%.

Of the 25 tests in Table 11, none has a saturation of as much as 90%, 5 have a saturation value of 85% to 89% and 20 less than 85%.

Conditions found in Tables 10 and 11 indicate excessive amounts of air space, a condition that indicates inadequate compaction. This statement is verified by the lowered strength indicated where identical samples with a greater total pore space or a higher per cent of saturation is found.

Many observations made during this research on total pore space have led to the belief that when a value of 40% or more has been reached, one should expect the strength to be too small for use in a roadbed. Moreover, when a roadbed is very weak, error in testing may not be accurately reflected by total pore space and its per cent of saturation. This seems to be indicated by several low-strength tests in Tables 10 and 11.

Table 12 has been compiled to show whether or not it offers a qualitative support to Tables 9, 10, and 11. The data in Table 12 show that the sources from which the data are obtained seem to indicate that when total pore space approaches a value of about 30%, quality is regarded as only fair or lower. When a value of 40% or more is found, one may expect poor to unsatisfactory conditions. Table 13 is a modification of Table 12 intended only for rough estimates of quality to be expected.

TABLE 11.—INTERRELATIONSHIP OF PORE SPACE, WATER CONTENT AND STRENGTH

Number	Density in grams per cubic centimeter		Percentage of water, by weight			Percentage of total pore space	Percentage of original saturation	Strength, CBR value
	Apparent	Bulk	Capacity	Content (original)	Content (future maximum[a])			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1a	2.67	2.08	10.6	7.5	9.5	22.0	70.7	105
1a	2.67	2.07	10.8	7.8	9.7	22.4	72.3	77
1b	2.67	1.98	13.0	10.0	11.7	25.7	76.9	27
1c	2.67	1.95	13.8	10.0	12.4	27.0	72.4	24
1b	2.67	1.96	13.5	9.8	12.1	26.5	72.6	20
23a	2.67	2.08	10.6	7.5	9.5	22.0	70.8	102
23c	2.67	2.02	12.0	9.0	10.8	24.3	75.0	32
23b	2.67	2.00	12.5	9.0	11.3	25.0	72.0	25
23d	2.67	1.97	13.2	11.6	11.9	26.1	87.9	16
20c	2.73	1.91	15.7	13.5	14.1	30.0	86.0	100
20a	2.73	1.63	24.7	19.5	22.2	40.3	78.9	40
20b	2.73	1.40	34.7	28.0	31.2	48.6	80.6	11
20a	2.73	1.63	24.7	20.0	22.2	40.3	81.0	2
20c	2.73	1.92	15.4	13.5	13.8	29.5	87.6	2
18c	2.74	1.93	15.2	13.0	13.7	29.5	85.5	62
18c	2.74	1.94	15.1	13.0	13.6	29.2	86.0	30
18a	2.74	1.66	23.7	15.0	21.3	39.4	63.2	14
18a	2.74	1.84	17.6	14.6	15.8	32.8	82.8	12
18b	2.74	1.80	19.1	15.3	17.2	34.3	80.0	10
18b	2.74	1.72	21.6	18.0	19.5	37.2	83.2	6
3a	2.74	1.78	19.7	15.5	17.7	35.0	78.5	22
3a	2.74	1.77	19.9	14.0	17.9	35.3	70.4	18
3b	2.74	1.67	23.4	17.7	21.0	39.0	75.5	13
3c	2.74	1.72	21.6	18.0	19.5	37.2	83.3	11
3b	2.74	1.66	23.8	18.0	21.4	39.4	75.5	7

[a] Percentage water is based on dry weight. Probable future saturation is based on the assumption that pore space is likely to become 90% saturated. The letter following the sample number designates the method of compaction; the letter "a" indicates the modified Proctor method; the letter "b", the standard Proctor method; the letter "c", the Porter static method and the letter "d", the 15-blow Proctor method. Future strength will be lessened only when Col. 6 exceeds value in Col. 5.

Additional Examples of the Porosity-Strength Relationship.—Table 4 supplements data in Tables 9, 10 and 11. Of these, examples of the Omaha Airport after the flood of 1943 had covered it during a period of 10 to 17 days without failure, are impressive. Data on this example are shown in Items 9 to 21. Data on siltstone (compacted silt), Item 1, and unconsolidated silt, Item 14, exemplify the effect of compaction on the resistance of silt under the conditions cited.

TABLE 12.—QUALITY OF ROADBEDS AS RATED BY PORE SPACE

No.	BPR classification	Percent pore space							Quality
		BPR[a]	Ohio[b]	Maryland[c,d]	North Carolina[e]	Iowa[d,f]	Indiana[d,g]	Other States[d,g]	
1	A-1	21-	....	21.5	18-25	....	....	....	Excellent
2	...	....	21-26	....	....	....	....	....	Good
3	A-2	21-27	....	26.9	19-27	....	....	....	Good
4	A-3	21-27	....	....	....	....	....	....	Good
5	...	29-32	....	....	....	....	....	....	Fair
6	A-4	27-34	....	30.6	25-34	....	....	....	Good-Poor
7	...	....	....	....	....	37.4	....	....	No frostboils
8	...	....	....	....	....	44.0	....	....	Frostboils
9	A-5	40-52	....	....	30-45	....	....	....	Poor-Very poor
10	...	....	25-38	....	....	....	....	....	Poor
11	...	....	....	....	....	....	35.0	....	Pumping
12	....	....	....	....	....	....	....	41.7	Pumping
13	A-6	34-52	....	30.7	....	....	....	....	Fair-Poor
14	A-7	34-52	....	34.8	36-51	....	....	....	Fair-Poor
15	A-8	45+	....	....	....	....	....	....	Unsatisfactory
16	...	....	41-49	....	....	....	....	....	Unsatisfactory

[a] Public Roads Administration, Principles of Highway Construction, pp. 60-61, Table 6, 1943. Fractional percents left off. [b] Woods & Litehiser, Summary of Test Averages of 467 Ohio Soil Samples, Ohio Eng. Exp. Sta. Bull. 99, p. 42, Jul. 1938. Fractional percents left off. [c] Highway Research Bd., Road Test ONE-MD, Highway Res. Bd., Spec. Rept. 4, 1952, Table 19. [d] Values are average values. [e] Hicks, L.D., Observations of Moisture Contents and Densities of Soil-Type Bases and Their Subgrades (NC), Proc. Highway Res. Bd., Dec. 6-10, 1948, p. 492. Fractional percents left off. [f] Mullis, Ira B., Design of Roadbeds, Trans. ASCE, v. 111, (1946), pp. 699-742, Table 12. [g] Woods, K.B. and T.E. Shelburn, Report of Committee on Maintenance of Concrete Pavements as Related to Pumping Action of Slabs, Proc. Highway Res. Bd., Natl. Res. Council, v. 23, 1943, p. 309.

Rate of Saturation of Earthwork.—Rate of saturation must depend on a number of variables, including continuity of pores, number and size of openings and perhaps other variables. For example, a freshly cultivated field becomes thoroughly soaked after a few hours of rain, but a well-trodden and puddled barnyard or rigid sand-clay road surface offers enormous resistance to complete saturation (Tables 2 and 4).

In only one case has a measured rate of saturation come within the scope of this research. In December, 1937, the Division of Tests and Research, Iowa

State Highway Commission, under the direction of Bert Myers, excavated five pits near its laboratory in Ames to depths of 48 in. and backfilled them with tamped earth of various types. Two pits contained loess, one contained glacial clay and the remaining two, referred to as pilot pits, were made up of earth excavated from the pits. Through the courtesy of James Johnson of the State's Research Department, five sets of tests, the last one on June 9, 1941, have been made available for this research as follows: The two loess pits with an average pore space of 36% that remained constant during nearly 4 yr at the depth 24 in. to 48 in., increased in saturation from 72% to 91%; glacial clay through the same period and depth, with an average pore space of 26% that remained constant increased in saturation from 62% to 83%; one pilot pit during the same period and depth with an average pore space of 40% that remained constant increased in saturation from 30% to 73%; the remaining pilot pit during the same period and depth had an average pore space that remained constant at 40%, increased in saturation from 40% to 61%. Computation of percentage pore space and saturation is the sole responsibility of this paper, as determined from the density and moisture measurements obtained from the source mentioned.

TABLE 13.—TENTATIVE CLASSIFICATION OF ROADBEDS,
FOR PRELIMINARY WORK ONLY

Density Apparent	Bulk	Total pore space	Voids ratio	Quality
2.68	2.54	5.0	0.05	Excellent
2.68	2.41	10.0	.11	Excellent
2.68	2.28	15.0	.18	Very good
2.68	2.14	20.0	.25	Good
2.68	2.01	25.0	.33	Fair
2.68	1.88	30.0	.43	Fair
2.68	1.74	35.0	.54	Poor
2.68	1.61	40.0	.67	Very poor
2.68	1.47	45.0	.82	Unsatisfactory

In all of the pits the increase in saturation is shown to be reasonably constant and, on the basis of findings already presented, it seems that the loess pits had probably approached their limit; on the same basis it seems probable that the glacial clay with a slower rate of increase might be expected to continue for some time before approaching its limit; the two pilot pits also remained considerable below the maximum saturation value found in certain rocks shown in Table 2 or the roadbeds of the Omaha Airport (Table 4).

Capillary Experiments.—Supplemental experiments on capillary action made too late to be included in a report made by the writer(30) using the method described by Lynde and Dupre completely refutes the belief that capillary water can accumulate under surfaces that restrict or prevent evaporation. The supplemental findings of the report mentioned are as follows:

1. When the funnel containing the sediment undergoing the capillary test was covered by a pane of glass that prevented evaporation, the rise of the water column stopped at once. Upon removal of the cover, upward movement of the column of water started at once.

2. When an electric fan was directed toward the surface under capillary test, of course, evaporation was accelerated; hence, capillary rise was also accelerated. When the fan was turned away from the wet surface of the sediment, capillary movement resumed its normal rise. In humid weather, of course, the rate of evaporation was reduced; hence the rate of capillary rise was correspondingly reduced.

3. When a few drops of water were placed on the sediment under test, the column of water dropped in proportion to the volume of water placed on the sediment. When a sufficient volume of water was added to the sediment, complete drainage of the column of water followed. When complete drainage was produced, air entered the system and prevented another capillary rise until air had been removed from the system.

It is, therefore, apparent that capillary action is dominated by air, evaporation, and suction of the sediment. The conditions cited seem to invalidate the belief that water from capillary action can accumulate under a pavement.

Frost Action.—During this research special attention was given to the effect of frost action in parts of the United States where it causes roadbed failures. Of the many examinations made during the periods of freezing, the outstanding areas included are the central part of Maine, Rhode Island, Ohio, northern Iowa, and northern Minnesota. Although other localities have also been examined during periods of frost action, these will serve as examples of severe conditions.

Without reciting numerous details contributing to frost action in roadbeds, only three conditions seem essential to the formation of soft spots due to frost. These are porous roadbeds, water, and freezing temperatures. This leads to the further statement that any increase of pore space, water content, or lowering of temperature increases the magnitude of the action. Obviously, the number of cycles of freezing and thawing increases the pore space, provided water is available for filling the pore space during thaws. A common source of water is melting snow on higher ground without intercepting drains. Another condition favorable to frost action occurs when a soaking rain is quickly followed by a deep freeze. This is a common observation made by many.

Conditions favorable to high porosity are silts, sand, organic matter, mica flakes, diatomaceous earth, unconsolidated beds of clay, etc. Areas of these beds are well-known locations of frost action. Frost action is also induced by the absence of wind or the presence of shade.

Conditions unfavorable to frost action are consolidated beds made up of any type of earthy matter protected against cycles of wetting and drying or other causes that disintegrate the surface. In no place included in this research has a soft spot or frost boil been found in consolidated beds.

A valid reason for consolidated beds failing to heave under freezing temperatures is due to limited pore space, rarely filled with water. The remaining space is, of course, filled with air and the water and air content resist the entrance of additional water. However, in case pore space is 90% filled with water and freezing occurs, expansion due to freezing being only about 9%, the pore space becomes only 99% filled. Of course, no heaving occurs until the pore space becomes filled. It should be observed, however, that pore space in rocks and sediments rarely becomes 90% filled as shown in Tables 2, 4, 9, 10, and Reference 8.

In support of the statement that frost action is reduced to the minimum or entirely lacking in rocks and sediments of very limited pore space, it was observed that a very dense rock formation near Duluth, Minn., had been planed off and deeply grooved by glacial action, but appeared to be fresh as though the action had been of recent occurrence. Here temperatures of -60° F are not uncommon.

In contrast to the foregoing statement, L. Don Leet and Sheldon Judson(32) say that obvious requirements for destructive frost action are a supply of water, a porous host material, and temperatures that move back and forth across the freezing line. The authors also give an example of the marked effectiveness of frost action in spalling chips from a sandstone monument in the Burying Ground of Christ Church, Cambridge, Mass. (The porosity of building sandstones is said to be 6.9% to 25.5%(6)). Additional support is given by Longwell and Flint in a statement saying that freezing of water held in open spaces of rocks and mantle is commonplace in all temperate and colder regions.(33) They too illustrate the spalling effect of weather causing a sandstone column beside a door—way of Durham Castle, England, after an exposure of weather for 263 yr.

As capillary action is increased by low temperatures, this action sometimes adds to the thickness of layers of ice at the surface; however, no such accumulation seems possible when total pore space below is reduced to amounts specified in the paper. Of course, much breakage has been found where the most solid rocks are wedged apart by the action of freezing water in joints and cracks, though not otherwise. As already stated, very porous rocks such as certain sandstones or limestones are often damaged by frost action after the total pore space has been filled with water. One case has come to the attention of this research in which an argillaceous limestone had been reduced to mud after repeated freezing and water-thawing in a laboratory. A portland cement concrete pavement made of this stone underwent severe disintegration also.

COMPACTION OF ROADBEDS

The basis of this paper rests on the fundamental fact that rocks and sediments of all kinds, regardless of origin or environment, are strong and resistant to water and frost action only when their pore spaces are small, contain little or no water, and have the minerals cemented together with a strong substance. Moreover, the premises set up in the beginning of the paper should be regarded as amplification of the fact just stated. Every test recorded in the tables is intended to exemplify details of certain principles that contribute to the strength of the rock or sediment it represents. Tables 9, 10, and 11 are especially useful as examples of the effect of total pore space and water content on strength. Many additional tables are available and all support the premises mentioned. However, a close examination of Table 11 will show that when values are close, the effects of total pore space may lead one to conclude that it is in error. Although the effect of non-homogeneity has already been mentioned, the reader is again cautioned about the fallacy of depending on a single test. Safety rests only in numbers, a fact well-known among testing engineers.

Compaction Procedure.—As the basis of the paper is intended to rest only on fact, every effort should be made to determine whether the basis is or is not fact. This can best be done through the application of the scientific method, also known as "The Method of Multiple Working Hypothesis".(31) The method consists in arranging known and related facts according to a definite idea which shall explain them all consistently. Safely lies in numbers and if one hypothesis, and one only, has met all tests that one could devise, then is one not justified in adopting that hypothesis as true? In accordance with this method the procedure suggested is as follows:

1. Subject each of the premises on which the paper is based to every test that can be devised;

2. Scrutinize the interrelationship of items in Cols. 4 to 9, Tables 9, 10, and 11 and compare the findings with other parts of the paper in an effort to find harmony or discord;

3. Make strenuous effort to determine whether the premises are or are not in harmony with the properties of minerals, rocks and sediments; and

4. If a premise is believed to be in error, it should be regarded only as a hypothesis and tested until truth is established. As truth or fact is the only authority in the field of science, one should refrain from the acceptance of source as proof of fact.

Having followed the process suggested, one should now be in a position to test the validity of the information received from any source.

Laboratory Tests.—As the equipment required is simple, it seems probable that most road laboratories will need little or no extra equipment. Under the definition of roadbeds, the pavement base, where needed, is included; hence, crushed rock or gravel should be available. Moreover, the lower part of the roadbed may include coarse material. For this reason two sizes of cylinders will be required. For fine and coarse sediments it is desirable that compaction and testing cylinders for 2-in. and 6-in. specimens be provided. Cylinders are identical to those in use for the Hubbard-Field stability test for asphaltic mixtures. Other cylinders now in use, including those in triaxial tests may also be utilized. Tampers, such as those used with the Hubbard-Field apparatus are satisfactory. Strength tests may be in the form of Hubbard-Field stability, compression or triaxial tests. As the compacted specimens are never dry in the roadbed, they should, of course, be tested with the water contained at the time of formation.

Specimens for strength tests are expected to conform to Premises 10, 12, 13, 15 and 16. These and other premises set up are believed to give adequate reasons for the several factors embodied in the specimens.

Premise 16 is believed to give ample reason for the reduction of total pore space. Under this premise the percentage of total pore space must be restricted to the value required to resist a certain specified load.

Occasionally one hears the statement that fine sediments in roadbeds may be compacted too much for ideal results. As yet, no adequate reason has been found that seems to support the belief. For examples, shale (Table 4, Nos. 2, 3, 5 and 6) is more resistant to load and weather than the average clay with a total pore space of 35% (Table 3, No. 17). The principal difference between clay and shale is that of total pore space and water content, especially the less compacted values that have not lost their property of plasticity.

As some may doubt the validity of Premise 15, fact may be established by porosity-strength tests needed on all types of roadbed materials. To determine this relationship, specimens of high, low and medium total pore space, as in Premise 13, should be prepared and subjected to the strength test with its water content at the time of compaction. Of course, strength values may be extended by the well-known method of interpolation or extrapolation. To give confidence, extended values should be verified by actual test. Obviously, the actual determination of the porosity-strength relationship when saturation is about 90% complete is a must under this method. To determine the resistance a given type of material offers to the entrance of water when it is less than 90% saturated at a given total pore space, its tolerance should be determined for convenience and also for economy. Of course, the entrance of water can be determined by weight and strength.

The preparation of pavement bases follows close to the procedure given for fine materials, however, much of the work of reduction of total pore space must, of course, be accomplished by the gradation of the materials. As the principles of gradation are well-known, discussion seems out of place.

Sedimentary rocks, including consolidated specimens of clay, offer greater resistance to load and weather than do their more porous and wetter equivalents. It is therefore suggested that strength tests be made on moist specimens of fine cohesive sediments already described. Tampers should be of such shape and applied with such force that total pore space and air content will be reduced to very small values. Layers of earth must be thin enough to allow ample escape of air so that its volume will be not more than 10% of the final total pore space. No thought, in the laboratory, should be given as to whether or not contemporary rollers can duplicate the results obtained in the laboratory. This is a separate problem for the designer of field compaction equipment. Once the designer of roadbeds becomes fully aware that a certain minimum pore space and water content are essential to the desired strength of the roadbed, experience shows that when the demand becomes sufficiently urgent, some wide-awake manufacturer will produce the equipment.

Pavement Base.—In the preparation of this paper great effort has been made to construct each layer of the roadbed to the economic limit of its capacity for strength. No layer should be expected to compensate for the weakness of an underlying layer. If this practice is followed, the use of more expensive material brought from a distance can be lessened or eliminated. Under the most favorable conditions a local material may be made adequate for the top or protective surface. However, in the absence of suitable local aggregates, these must be provided from other sources.

Again, a limited total pore space and a strong bond are essential. To provide these, suitable materials must be found and laboratory work must determine the gradation of the aggregates and the particular deposit of clay to provide the strongest bond. Bond strength should, of course, be measured. In the beginning the most probable source is a brickyard where clay is already ground and ready for mixing. Preliminary aid in the selection of a suitable clay is given in Tables 5, 6, 7, and 8 and additional data under the headings "Properties of Clays," "Classification of Clays," and "Strength of Clays."

Proportioning the aggregates and binders is preferably by weight and mixing should be done by a machine and using a water content that will facilitate the thorough covering with a thin coating of clay. The mixed material should, of course, be allowed to aerate until the water content allows the material to be rolled until it attains the pore space planned. Total pore space should be not more than 10%. By careful planning and work it can be reduced to lower values.

The pavement base should extend to within 1 ft of the edges of the roadway for added strength to the top of the roadbed and as soon as it becomes sufficiently dry for asphalt to adhere and before traffic is permitted, one or more applications of liquid asphalt should be applied and blotted with sand to prevent ravelling and evaporation. Where traffic requires a hot asphaltic mix, it should be applied.

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DISCUSSION

JACOB FELD,[3] F. ASCE.—The author's approach to the preparation of foundations for pavements is as an engineering analysis of soil as a construction material, common practice prior to the popularization of intricate and detailed laboratory investigation of what were assumed to be representative samples of the soil. Listing premises in simple terms, the subject is made clear and, generally, cannot be criticized. Of course, there are many minor points of disagreement, and many examples can be cited of actual constructions that refused to follow the simple pattern of action listed in the paper. But the chief reason for such non-agreement is the fact that even uniform soil is not as uniform as a man made material or even a natural material like timber; furthermore, soil has the habit of changing its characteristics with only slight variation in moisture or in included impurities. Metal alloys have the same habit, but once the change is frozen into the solid mass, with a few exceptions of effect from vibrations, the metal remains physically stable.

At the Highway Research Board meeting in January, 1948, the late C. A. Hogentogler, then Chairman of the Soils Division, and the writer, argued at some length on the advisability of correlation with known previous experience in the action of soils, not at the complete elimination of soil tests, but giving more weight to the results of actual experience than to the results of laboratory tests. The large expansion of soil laboratory techniques and technicians at that time outweighed reports of actual job results that were being presented to the Board. At the following meeting (December, 1948) of the Highway Research Board, the writer presented a paper on "General Engineering Approach to the Classification and Identification of Soil," in which the thesis was expressed as:

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> "The purpose of a soil classification and identification system for engineering control and use of soils is to permit the extrapolation of known physical actions of some soils and thereby prophesy the action of other soils under similar exterior loadings, temperature and moisture conditions. The answers desired are physical strains and stresses. The question must include that possibility. The proper classification must therefore be, not on size, color, taste or feel, but on the basis of numerical physical characteristics."

Included in that paper are references to similar approaches in the engineering use and control of other materials.

The author's approach to the problem is along similar lines, and the student steeped in soil mechanics may be shocked to find that engineering structures can be designed without extensive laboratory testing.

However, this is no reason for any reduction in the fine basic studies of soil structure and action carried on in the laboratory. Much has been learned of why certain soils act as they do from scientific ivenstigations of samples. One of the very fruitful examples is the X-Ray analysis of fine grained soils. W. L. Bragg,[4] in summarizing British contributions in this field, notes his identification of four distinct patterns of oxygen-silicon-aluminum ion arrangements that he compares to purl and plain stitches in knitting. The four great families of the silicate minerals are arrays of terahedra made up of four oxygen atoms, octahedra of six aluminum atoms (like two pyramids joined base to base), and single pyramids of four silicon atoms. The patterns built up are separate units of the terahedra (dense olivines), joined corner to corner in strings (asbestos), joined by three corners to make sheets (mica) and joined by all corners to make a three dimensioned framework (feldspars). It must be admitted that no general approach to the soil problem could by observation or deduction bring this information to light. Nor can it be argued, on the other hand, that such fine scientific research by itself can be the tool for engineering design.

Only a proper use of each approach to answer the questions set for each, can result in further advance of the quest for safer and more economical engineering designs. In bringing to attention the broad practical approach, the author has contributed a worthwhile paper.

W. H. CAMPEN,[5] F. ASCE, L. G. ERICKSON,[6] AND J. R. SMITH.[7]—Although Mr. Mullis includes much more or less related data in his paper, he has two main objects in view. First, that the strength of compacted road beds be measured by some suitable method instead of being judged by density, and second, that the strength be correlated with pore space and moisture content in order to obtain permanency. The writers are in general agreement with the basic principles involved, but differ in some respects, for instance, the method of testing for strength, the importance of bond in the attainment of strength, the pore space limitation, and the requirement for the percent water-filled pore space. In addition, this discussion will indicate how density may be used during construction to control the desired properties.

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[4] Science, Vol. 131, June 24, 1960, p. 1870.

[5] Pres., Omaha Testing Labs., Omaha, Neb.

[6] Omaha Testing Labs., Omaha, Neb.

[7] Omaha Testing Labs., Omaha, Neb.

The writers have reported[8] the results of an investigation of six typical fine grained soils and six soil-aggregate mixtures. Each of the samples was compacted by three methods, the standard Proctor, the modified Proctor method and a method with intermediate compactive effort. The methods deliver energies of 177, 322, and 483 ft-poundals per blow and 235, 428, 965 ft-poundals per cu in. of soil. The strength was measured by a method similar to the California Bearing Ratio (CBR) Method but determined at 1/4 in. pennetration and in pounds per square inch.

TABLE 15.—MAXIMUM DENSITIES AND OPTIMUM MOISTURES WITH DIFFERENT METHODS OF COMPACTION

Soil No.	Densities and Moistures					
	Method No. 1		Method No. 2		Method No. 3	
	Density, in pcf	Moisture, in percentage	Density, in pcf	Moisture, in percentage	Density, in pcf	Moisture, in percentage
1	113.5	15.0				
2	110.0	18.0	112.5	17.0	116.2	15.7
6	107.5	18.1				
9	124.2	10.5				
10	113.7	15.0	117.5	13.2	124.3	10.6
13	101.0	18.5	106.6	17.9	109.2	16.0
Soil Aggregate No.						
1	139.5	6.0				
2	137.5	6.7				
7	140.8	5.8	141.5	5.0	143.5	3.7
8	138.9	5.9	140.4	5.3	142.9	4.3
9	141.9	4.8	142.2	4.8	142.4	4.4
11	130.0	9.4	130.2	8.9	135.2	6.9

The characteristics of the soils and mixtures are given in Table 14. The maximum density and related data are given in Tables 15 and 16. The strengths for all the soils and mixtures, after compaction to maximum density by the standard method, are given in Table 17. The strengths of three soils and four soil aggregate mixtures after compaction to maximum density with the three methods are given in Table 18.

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[8] "The Bearing Index as a Criterion for the Maximum Density Requirement," by W. H. Campen, L. G. Erickson, and J. R. Smith, Highway Research Bd., 1945.

TABLE 14.—CHARACTERISTICS OF SOIL MIXTURES

Soil		Gradation—Passing Sieve No. in percentage						Liquid Limit	Plasticity Index	Spec. Gr.
No.	Source	270	200	40	10	4	3/4-in.			
1	Omaha, Nebr.	84.0	88.0	100.0	100.0	100.0	100.0	29.5	5.0	2.69
2	Omaha, Nebr.	98.0	100.0	100.0	100.0	100.0	100.0	34.5	10.5	2.7-
6	Kearney, Nebr.	85.0	97.5	100.0	100.0	100.0	100.0	38.0	19.0	2.68
9	Grand Island, Nebr.	48.0	51.4	80.0	100.0	100.0	100.0	24.0	9.0	2.62
10	Waterloo, Ia.		34.6	67.6	94.4	100.0	100.0	24.2	7.7	2.65
13	Scribner, Nebr.	96.8	98.6	99.8	100.0	100.0	100.0	41.7	21.1	2.60
Soil-Aggregate										
No.	Source									
1	Omaha, Nebr.		20.0	30.0	60.0	88.0	100.0	26.5	7.5	2.63
2	Omaha, Nebr.		20.0	30.0	60.0	88.0	100.0	34.0	21.0	2.62
7	Waterloo, Ia.		7.2	24.7	42.5	47.3	100.0	15.2	2.8	2.63
8	Sheridan, Wyo.		13.8	26.2	52.3	61.6	100.0	24.5	13.0	2.60
9	Omaha, Nebr.		11.4	26.3	47.3	71.9	100.0	20.1	4.8	2.61
11	Waterloo, Ia.		11.2	30.1	75.1	91.7	100.0	23.0	5.0	2.64

The relationship between strength and maximum density for three of the fine grained soils is shown graphically in Fig. 1. The density-energy relationship is shown in Fig. 2, and the strength-energy relationship is shown in Fig. 3.

A study of the data in Tables 17 and 18 warrant the following deductions:

1. The strength of soils and soil-aggregate mixtures compacted to maximum density at optimum moisture by the standard Proctor method varies over a wide range (Table 17) and does not appear to be directly related to gradation, plasticity index, unit weight, or pore space. This substantiates Mr. Mullis' contention that the maximum density is not a good criterion for strength.

TABLE 16.—VOLUMETRIC

Soil No.	Compaction Method #1					Compaction	
	Percentage by Volume					Percentage	
	Solids	Water	Air	Pore Space	Pore Space Filled	Solids	Water
1	67.62	27.28	5.10	32.38	84.2		
2	65.29	31.73	2.98	34.71	92.1	66.77	30.65
6	64.05	31.18	4.77	35.95	86.8		
9	75.97	20.90	3.13	24.03	87.0		
10	68.76	27.33	3.91	31.24	89.0	71.06	24.86
13	62.25	29.94	7.81	37.75	82.9	65.70	30.58
Soil Aggregate No.							
1	85.00	13.41	1.59	15.00	89.4		
2	84.10	14.76	1.14	15.90	92.8		
7	85.80	13.09	1.11	14.20	92.1	86.22	11.34
8	85.61	13.13	1.26	14.39	91.3	86.54	11.92
9	87.13	10.92	1.95	12.87	85.0	87.31	10.94
11	79.52	19.58	0.90	20.48	95.6	79.64	18.57

The data in Table 18 show that the strength, with any one soil or soil aggregate mixture, increases as the density is increased. However, the increase in strength is not directly proportional to density when all the soils or soil-aggregate mixtures are considered. This is further proof that the strength can not be judged from the density. This point is further emphasized by the curves in Fig. 1, that are constructed from the data in Table 18 for three fine-grained soils.

The curves in Fig. 2 show that the energy required to compact soils is related, in a general way, to the plasticity index, in that the higher the plasticity index the greater the amount of energy required. These data are not directly related to this discussion but are included because it is related to the general subject of soil-densification.

The curves in Fig. 3 show the relationship between energy and strength. They indicate that the amount of energy required to produce a given strength varies over a wide range. For instance, if a strength of 478 psi is required, the energy per cubic inch of compacted material is 410, 490, and 665 ft-poundals for soils 13, 10, and 2 respectively. The principal reason for including these data is to point out that density, while not a quantitative measure of strength, can be used to control strength. Thus, after the density required to produce desired strength with a given soil has been determined, it can then be used to control field compaction.

2. The data in Tables 15, 16, 17 and 18 can also be used to discuss the effect of bond strength, pore space, and percentage of water-filled pore space. First, it is evident that bond does not control strength. Soil sample 13, Table 17, has the highest plasticity index, that is, a measure of bond strength, but its strength is low. Sample 1, with very low plasticity index, has excellent strength. In the same table, soil-aggregate mixtures 9 and 11 have equal plasticity index, but the former is 8 times as strong as 11. Furthermore, samples 7 and 8 have similar strength but entirely different plasticity indexes.

ANALYSES

Method #2			Compaction Method #3				
by Volume			Percentage by Volume				
Air	Pore Space	Pore Space Filled	Solids	Water	Air	Pore Space	Pore Space Filled
2.58	33.23	92.8	68.97	29.24	1.79	31.03	94.60
4.08	28.94	87.6	75.17	21.12	3.71	24.83	87.00
3.72	34.30	89.8	67.31	28.00	4.69	32.69	87.40
2.44	13.78	82.2	87.44	8.51	4.05	12.56	67.80
1.54	13.46	88.6	87.77	10.04	2.19	12.23	82.00
1.76	12.69	86.2	87.44	10.04	2.52	12.56	80.00
1.79	20.36	91.2	82.70	14.95	2.35	17.30	86.4

In connection with this matter of bond strength, it should be pointed out that clean sands and gravels, whether graded or not, possess very high bearing values when thoroughly consolidated. Furthermore, well graded cohesionless mixtures of rounded or angular materials possess high bearing values also.

3. As far as pore space is concerned, all the fine grained soils, Table 16, have pore spaces ranging from 24% to 38%, when compacted by the standard method, and when soils 2, 10, and 13 are compacted by the modified method they have pore space content ranging from 25% to 33%. All of these pore space contents show that normal typical fine grained soils do not have more than 40% pore space after compaction.

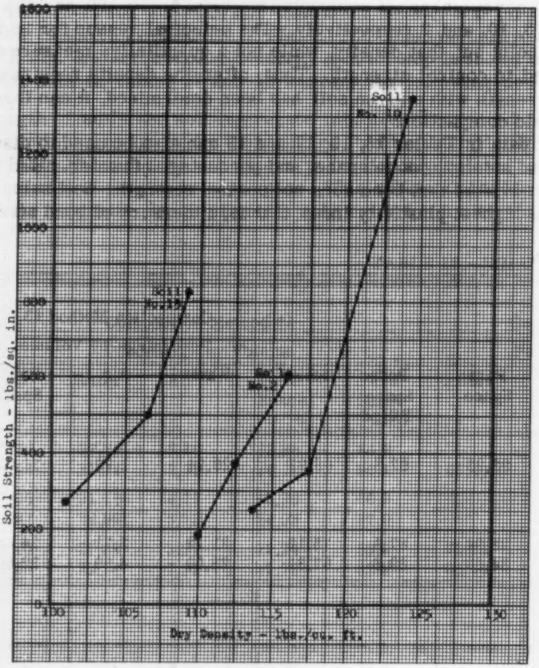

FIG. 1.—DENSITY-STRENGTH RELATIONSHIP

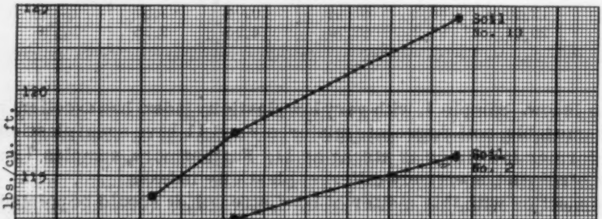

FIG. 2.—DENSITY-ENERGY RELATIONSHIP

Looking now at the soil-aggregate mixtures in Table 16, it will be noted that the pore space ranges from 13% to 16%, with the exception of sample 11, that shows 21%, after compaction with the standard method. Furthermore, when samples 7, 8, 9, and 11 are compacted by the modified method, the pore space ranges from 12.0% to 12.5%. All of these tests indicate that a maximum pore space of 10%, as required by Mr. Mullis, is not feasible. At this point, attention should be called to the fact that the modified method of compaction involves a compactive effort, that is extremely difficult to duplicate in the field.

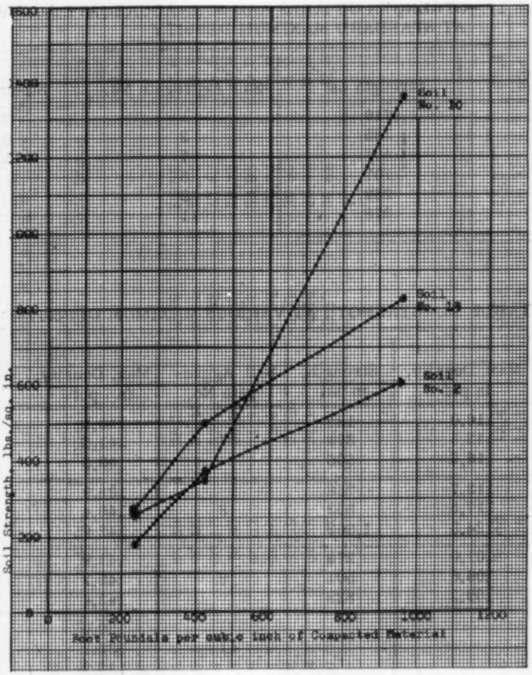

FIG. 3.—COMPACTIVE EFFORT-STRENGTH RELATIONSHIP

Mr. Mullis' requirement for filling 90% of the pore space with water during the densification process is not only too rigid but is not realistic when compared with actual results. A study of the results obtained with the fine grained soils, Table 16, shows water-filled percentages ranging from 85% to 92% when compaction is done by the standard method. When the modified method is used the percentage ranges from 87% to 95%.

The results shown for the soil-aggregate mixtures, Table 16, support Mr. Mullis' contention in five out of the six samples when compaction is done by the standard method. However, when higher compactive efforts are used, in order to develop less pore space and more strength, the percent of water-filled pore space decreases. This phenomenon is not readily explainable, but the results are not to be questioned. It appears that in order to attain high percentages of water-filled pore space, the densities have to be reduced. This will increase the percentage of water and usually reduce the strength.

TABLE 17.—STRENGTH OF SOILS AND SOIL-AGGREGATE MIXTURE AT STANDARD MAXIMUM DENSITY

Soil No.	Strength psi	Soil Aggregate No.	Strength psi
1	710	1	334
2	185	2	643
6	407	7	137
9	707	8	111
10	258	9	1,273
13	278	11	159

TABLE 18.—STRENGTH AT VARIOUS MAXIMUM DENSITIES

Soil No.	Max. Density, in pcf	Strength, in psi	Soil Aggregate No.	Max. Density, in pcf	Strength, in psi
2	110.0	185	7	140.8	137
	112.5	376		141.5	834
	116.2	605		143.5	2,419
10	113.7	258	8	138.9	111
	117.5	353		140.4	390
	124.3	1,362		142.9	573
13	101.0	278	9	141.9	1,273
	106.6	501		142.2	1,416
	109.2	828		142.4	1,974
			11	130.0	159
				130.2	185
				135.2	1,054

Mr. Mullis asserts that probably 90% of the pore space will eventually become filled with water. The writers do not have field data along this line but do have some laboratory data. In connection with the determination of volume change on drying and freezing, that will be discussed hereinafter, the absorption by capillarity of compacted samples was determined, and in order to show the effect on the saturation, the percent of water-filled pore space was computed. The results obtained with 9 soils and 5 soil aggregate mixtures are shown in Table 19. It will be noted that the percentage of water filled space after absorption ranges from 87 to 95 for the fine grained soils and 90% to 99%

TABLE 19.—ABSORPTION BY CAPILLARITY AND RELATED DATA

Soil No.	Compaction Data		Volumetric Analyses—Percentage by Volume						
			Before Absorption					After Absorption	
	Dry Density, in pcf	Optimum Moisture, in percent	Solids	Water	Air	Pore Space	Pore Space Filled	Air	Pore Space Filled
1	113.3	15.5	67.50	28.14	4.36	32.50	86.6	2.05	93.7
2	125.0	10.2	76.46	20.43	3.11	23.54	86.8	2.67	88.6
3	111.5	16.5	66.18	29.49	4.33	33.82	87.2	3.79	88.8
4	115.3	13.9	70.80	25.69	3.59	29.20	87.9	1.98	93.2
5	110.0	18.1	66.02	31.91	2.07	33.98	93.9	1.54	95.5
6	105.8	17.5	64.96	29.67	5.37	35.04	84.6	2.83	92.0
7	115.9	14.7	70.09	27.30	2.61	29.91	91.2	1.96	93.5
8	107.5	18.1	64.28	31.18	4.54	35.72	85.0	3.25	90.8
9	105.5	20.0	63.32	33.81	2.87	36.68	92.2	2.19	94.0
Soil Aggregate No.									
1	139.5	6.0	85.00	13.41	1.59	15.0	89.4	0.94	93.8
2	136.0	6.5	83.19	14.17	2.64	16.81	84.4	1.33	89.5
3	140.3	5.8	86.15	13.04	0.81	13.85	94.2	0.16	98.8
4	137.0	6.9	83.80	15.15	1.05	16.20	93.5	0.52	96.8
5	138.6	5.9	85.43	13.10	1.47	14.57	89.9	0.60	96.0

for the soil aggregate mixtures. The results indicate that the degree of saturation of the soils and soil-aggregate mixtures varies over a fairly wide range, and of course, corresponding changes will occur in strength. It seems evident, therefore, that the proper procedure for design would be to compact soils and mixtures to as high densities as practicable to reduce pore spaces to practical minimums, and then to submit them to the action of capillary water before determining the strength.

The capillary tests were made by replacing the bottom of the casting mold with a perforated bottom, placing the mold assembly in a pan, adding water until 1/2 in. of the sample was submerged, and storing the pan in a moisture cabinet for 7 days. We believe the conditions of this test are at least as severe as those to be encountered by field installations.

Because Mr. Mullis mentions contraction and expansion in this paper, it is appropriate to include data[9] along those lines in this discussion.

Ten soils and six soil aggregate mixtures were studied by the writers. The plasticity index of the soils ranged from 5 to 52 and that of the soil aggregate mixture from 6 to 21.

The soils were compacted to near maximum density and optimum moisture by the standard Proctor method and were then expelled from the mold. Shrinkage on drying was determined by measuring the maximum change in diameter at room temperature. The volume change on freezing was determined by coating the plugs with paraffin wax and then submitting them to a temperature of 10° F below zero for 6 hr.

The results may be summarized as follows:

1. All mixtures shrink on drying and the shrinkage generally increases as the optimum moisture increases. The lineal shrinkage of the fine grained soils ranges from 0.5% to 4.3%, while that of the soil aggregate mixtures from 0.1% to 0.5%.
2. The fine-grained soils shrink on freezing, and the shrinkage increases as the optimum water content increases, whereas the soil-aggregate mixtures expand and the expansion increases as the ratio of water used at optimum to the computed optimum water content increases. The fine grained soils shrink from 0.1% to 1.0% and the soil-aggregate mixtures expand from 0.2 to 0.5%.

As has been noted, the strength of the soils in our research has been determined by a method similar to the CBR Method. In the writers' opinion this type of method is the proper one to use because it is suitable for all types of soil and soil-aggregate mixtures, from very plastic ones to noncohessive ones. The test measures resistance to consolidation and displacement, both of which determine the ability to sustain vertical loads. The Hubbard Field Test on the other hand, measures resistance to shear that is dependent on cohesion to a large extent. The test was not intended for cohesionless mixtures.

IRA B. MULLIS.[10]—The concept on which this research is based is that roadbeds can best be designed by the well-known rational method applied to the design of bridges and pavements. Moreover, as roadbeds must resist forces such as loads and weather, they must be rigid bodies; therefore, air and water must be restricted to very small percentages in the roadbeds.

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[9] "Some Physical Properties of Densified Soils," by W. H. Campen, L. G. Erickson, and J. R. Smith, Highway Research Bd., 1942.

[10] Cons. Engr. (retired), Raleigh, N. C.

Roadbed Design Based on Stress-Strain Method.—The discussion by Jacob Feld, F. ASCE plainly indicates that he recognizes the need for a rational method in the design of roadbeds, similar to that followed in the design of concrete mixes and steel bridges. The evidence of his hopefulness seems apparent when he mentions terms such as physical stresses and strains, extrapolation and other terms well-known to the engineer.

Concerning Feld's apparent hopefulness, it seems certain that a method for designing a roadbed to sustain certain loads and resist weather has been found. Although much effort has been made to find a condition in which the method fails to apply, no such condition has been found, and evidence of its value grows stronger as the research continues.

Tables 9, 10, and 11 are based on strength measured by two different methods. Results of the triaxial tests made by Gerald A. Leonards, M. ASCE (34) are shown in Table 20 and Fig. 4. The magnitude of the strength is arranged in decreasing order from the top to the bottom of the table. Density values, expressed in pounds per cubic foot in the original table, have been converted into grams per cubic centimeter to facilitate computations. Although the effects of pore space and saturation on compressive strength have been shown for Mississippi loess and Ft. Union clay, only that of the Mississippi loess is indicated. Each indicates similar trends; hence, only the Mississippi loess table is shown, though both loess and clay are shown in Fig. 4.

Maximum, minimum, and average values are shown for the Mississippi loess in Table 20. As total pore space and saturation do not have as wide range as oftentimes found in many tests, the table is well-adapted to the demonstration of the effects of total pore space and its saturation on compressive strength.

For plotting the line of average relationship of pore space when 90.3% saturated, saturation values between 88% and 92% only are included, but other values are plotted along the line with saturation values given to the nearest whole unit. This method of plotting is followed to give the most precise relationship between strength and pore space when 90% saturated. Saturation values much greater or less than 90% are given in the graph to the nearest whole number. Obviously, distance of points from the average 90.3% line indicates the precision of the tests.

A close examination of Fig. 4 shows that information may be obtained as follows:

1. Additional values of usable precision may be interpolated or extrapolated from the lower limit of proportionality almost to zero pore space.
2. By means of the graph, one may determine the lower limit of proportionality that may well be considered as the upper boundary region of excessive weakness.
3. In case the indicated strength of a specimen is based on a percent of saturation not located on the 90% line of the graph, higher or lower strengths may be determined by extrapolating from two or more points of identical saturation not on the 90% line.

For example, any of the three points on the Mississippi loess graph with a saturation value of 86% may be extrapolated to the 90% line above it. Also, as another example, the 69% and 70% points on the right side of the same graph also may be extrapolated either in an upward or downward direction. Obviously, pore spaces of different percentages of saturation will have different slopes also; consequently, this is a reason for the difference in slopes of the graphs of the Mississippi loess and the Ft. Union clay. Due to this fact, it should be apparent that pore space alone cannot indicate the true strength of a specimen. Also, as a given pore space attends a given bulk density only when attended

TABLE 20.—TRIAXIAL COMPRESSION TESTS ON MISSISSIPPI LOESS[a]

Series and No.	Density, in grams per cubic centimeter		Weight percent water		Pore space		Compressive strength in kilograms per square centimeter
	Apparent	Bulk	Capacity	Content	Total percentage	Saturation percentage	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
3- 9	2.72	1.77	19.3	17.7	34.8-	89.8	10.49+
3- 7	2.72	1.77	19.3	17.9	34.8	91.7	8.94
3- 3	2.72	1.75	20.4	17.8	35.6	87.2	7.05
1- 8	2.72	1.76	20.0	18.0	35.3	97.9+	6.90
2- 7	2.72	1.74	20.7	17.9	36.0	86.4	6.02
5- 9	2.72	1.72	21.3	19.9	36.7	93.4	5.55
3- 4	2.72	1.74	20.7	17.9	36.0	86.5	5.54
1- 7	2.72	1.72	21.3	18.1	36.7	89.1	5.46
3- 6	2.72	1.73	21.0	18.0	36.3	85.6	5.39
2- 5	2.72	1.74	20.7	17.8	36.0	86.0	5.24
3-10	2.72	1.72	21.3	19.1	36.7	89.6	4.68
3- 5	2.72	1.72	21.3	18.1	36.7	85.0	4.40
2- 4	2.72	1.68	22.8	17.5	38.2	76.7	4.20
5-10	2.72	1.68	22.8	21.0	38.2	92.0	3.72
5-14	2.72	1.68	22.8	20.9	38.2	91.6	3.62
1- 6	2.72	1.63	24.5	18.4	40.0	75.0	3.60
2- 3	2.72	1.62	24.9	17.5	40.4	70.2	3.05
5-15	2.72	1.66	23.5	21.0	39.0	89.2	2.98
5- 5	2.72	1.66	23.5	21.5	39.0	91.4	2.89
5- 7	2.72	1.64	24.2	21.9	39.7	90.5	2.32
2- 6	2.72	1.59	26.1	18.1	41.5	69.2	2.26
5-13	2.72	1.62	24.9	22.7	40.4	91.0	1.91
5-11	2.72	1.62	24.9	22.5	40.4	90.2	1.82
5- 6	2.72	1.61	25.4	22.8	40.8	89.7	1.62
1- 6	2.72	1.45	32.2	18.5	46.6+	57.4-	1.43
5- 3	2.72	1.59	26.1	23.4	41.5	89.6	1.32-
Average,		1.68	22.8	19.4	38.2	86.0	4.30

Maximum and minimum strength with its influencing values are shown by the signs + and - respectively.

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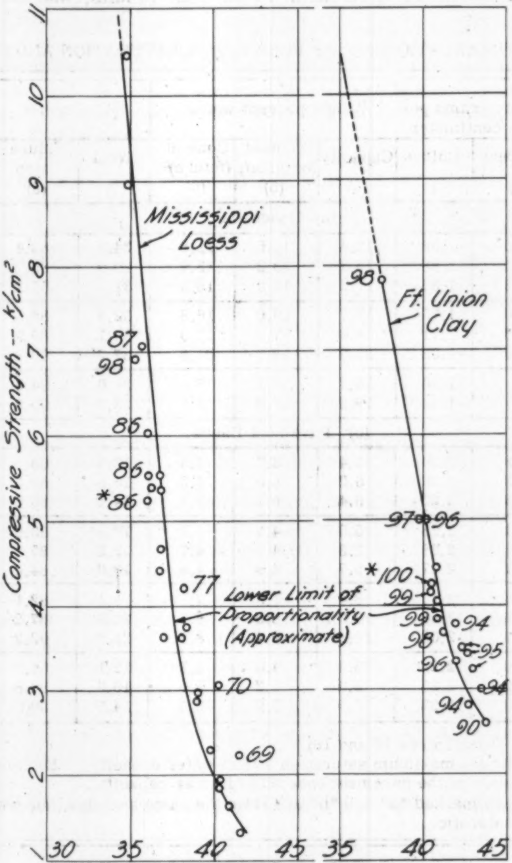
[a] Basic data, Cols. 1, 2, 3, 5, and 8 are from "Strength Characteristics of Compacted Clays," by Gerald A. Leonards, Transactions, ASCE, Vol. 120, 1955, Table 3. Maximum dry unit weight (Standard Proctor) 108.4 (1.74 g per cu cm). Maximum dry unit weight (Mod. AASHO) 120.7 pcf (1.93 g per cu cm). Optimum moisture (Standard Proctor) 18.0%. Liquid limit 37%. Optimum moisture (Mod. AASHO) 12.8%. Plastic limit 23%. Bulk density originally expressed in pounds per cubic foot, is here expressed in grams per cubic centimeter to facilitate computations. 1 kg per sq cm is equivalent to 14.2 psi. + = Maximum; - = minimum.

by a given apparent or mineral density, bulk density cannot be utilized as a true measure of strength.

Before closing the discussion of Table 20, it should be stated that less expensive apparatus than the triaxial apparatus can also be used for this work. Indeed, any accurate compaction mold and hammer of types now (1961) in use



NOTE: *Figures denote % Saturation

FIG. 4.—EFFECT OF PORE SPACE AT CONSTANT SATURATION ON COMPRESSIVE STRENGTH

may provide bulk density specimens for suitable strength measurements. For the computation of pore space, of course, apparent or mineral density is also essential. Any apparatus capable of giving accurate strength values is adequate.

Strength should be expressed in pounds per square inch or in kilograms per square centimeter. To save time in making computations bulk density should also be measured and expressed in metric units. Obviously, the strength of a roadbed should be in excess of the wheel load it is expected to support; therefore, the laboratory man should determine the greatest strength possible as shown in Fig. 4 and report the results. By this method, interpolation will disclose additional values of strength. From these results, final decision may be made. If compaction equipment is unavailable, perhaps it may be obtained. It will certainly not become available until the need for it is indicated.

TABLE 21.—INTERRELATIONSHIP OF PORE SPACE, SATURATION AND STRENGTH[a]

Specimen No.	Density grams per cubic centimeter		Weight percent water			Pore space		
	Apparent	Bulk	Capacity	Content (original)	Content (future)[b]	Total	Saturation	Strength, in psi
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
				(a) Roadbeds				
10	2.65	1.99	12.4	10.6	11.2	24.8	85.4	1,362
10a	2.65	1.88	15.4	13.2	13.9	29.0	85.7	353
10b	2.65	1.82	17.2	15.0	15.5	31.3	87.1	258
13	2.60	1.75	18.7	16.0	16.8	32.6	85.5	828
13a	2.60	1.71	19.9	17.9	17.9	34.1	89.9	501
13b	2.60	1.62	23.3	18.5	21.0	37.7	79.4	278
2	2.7	1.86	16.7	15.7	15.0	31.0	94.0	605
2a	2.7	1.80	18.5	17.0	16.6	33.3	91.9	376
				(b) Pavement Bases				
7	2.63	2.30	5.4	3.7	4.6	12.5	68.5	2,419
7a	2.63	2.26	6.2	5.0	5.3	14.0	80.5	834
7b	2.63	2.25	6.4	5.8	5.4	14.4	90.6	137
9	2.61	2.28	5.5	4.4	4.7	12.5	80.0	1,974
9a	2.61	2.28	5.5	4.8	4.7	12.5	87.2	1,416
9b	2.61	2.27	5.7	4.8	4.8	13.0	84.0	1,273
11	2.64	2.16	8.4	6.9	7.1	18.1	82.1	1,054
11a	2.64	2.08	10.2	8.9	8.7	21.3	87.2	185
11b	2.64	2.08	10.2	9.4	8.7	21.3	92.2	159
8	2.60	2.28	5.3	4.3	4.5	12.1	81.2	573
8a	2.60	2.25	5.9	5.3	5.0	13.3	89.8	390
8b	2.60	2.22	6.5	5.9	5.5	14.5	90.6	111

[a] (Basic data from Tables 17 and 18)

[b] Estimated future maximum saturation 90% of water capacity. Estimated future maximum saturation of the pavement base 85% of water capacity.

Note.—Numbers marked "a" and "b" are from the same samples, but with different pore space and saturation.

Concerning the so-called relationship between bulk density and strength, Campen and his associates say that density and strength are not proportional and cite Table 18 as an example. Table 21 is compiled, recomputed and rearranged in the order of decreasing strength from Tables 17 and 18. This table shows that neither bulk density nor pore space alone governs strength.

With regard to the pavement bases, attention is called to the effect of difference in saturation in Nos. 9 and 9a, 11a, and 11b.

Reduction of Pore Space Near Saturation Increases Strength.—The concept that increases in compaction of earthwork do not invariably produce an increase in strength seems to have been advanced first by Charles R. Foster, F. ASCE (35) then in turn by Willard J. Turnbull, F. ASCE and Foster (36), but later the concept was declared to be untenable by Harold J. Gibbs, M. ASCE and Jack W. Hilf, F. ASCE (37).

Under the heading, "Compaction of Roadbeds," the concept is again rejected with citations given showing that the reduction of pore space in a given material always tends toward an increase in strength. As pore space in a given material is lowered, of course, the solid content is increased; consequently, strength also is increased.

In an effort to show that reduction in soil strength does not follow increases in bulk density, data from Turnbull and Foster's Fig. 2 (36), entitled, "The Relationship Between CBR and Density for Various Laboratory Water Contents" are expressed in Table 22, Columns 1, 2, 4, and 8 and the remaining values, Columns 3, 5, 6, and 7 are computed therefrom.

To facilitate the comparison of values in Table 22, the data are arranged in the order of increasing molding water content and grouped according to pore space. As all specimens are made from the same sample, of course, the apparent density is constant. This being true, the pore space for a given bulk density must also be constant.

Table 22 is compiled from a graph said to show that a specimen of earth may, under certain conditions, be compacted too much for producing the desired strength. The outstanding properties of Table 22 are summarized as follows:

1. The strength of a specimen of earth containing a given percentage of water invariably decreases with increasing pore space.
2. The strength of a specimen of earth containing a given percentage of pore space invariably decreases with increasing saturation.
3. The strength of a specimen of earth is the greatest during wet or dry weather when its pore space is the minimum and its saturation has reduced pore suction to zero.
4. For most earthy matter, zero pore suction may well be regarded as approximately 90% saturation.
5. The least ability of a specimen of earth with a given pore space to resist loads during wet weather is when its pore space is entirely filled with dry air at the beginning of wet weather.
6. The greatest potential strength to be expected of any specimen in the table during wet weather is estimated to be one with values lying between Nos. 3 and 4. Such a specimen would have a pore space of about 32%, a saturation value of about 88% and a CBR strength of about 36%. During wet weather, the table indicates a CBR strength of about 23%.
7. The least potential strength to be expected of any specimen in the table is No. 33 with a pore space of 41.2%, a saturation of 46.5% and a CBR strength of 28%. During wet weather its soaked counterpart, No. 40, is shown by the table to have a CBR strength of 2%.
8. Although No. 4 with a molding water content of 16% is shown to be 90.9% saturated, its counterpart, No. 7, is shown to have a CBR strength of only 13% after soaking. The loss in strength from 20% to 13% seems too great. As

TABLE 22.—EFFECT OF PORE SPACE AND SATURATION ON THE STRENGTH OF ROADBEDS

No.	Density, in grams per cubic centimeter		Weight percent water			Pore space		Strength CBR, %
	Apparent	Bulk	Capacity	Content (molding)	Content (future)[a]	Total %	Saturation %	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	2.72	1.84	17.6	13	15.8	32.4	73.8	80 U[b]
2		1.84	17.6	14	15.8	32.4	79.5	70 U
3		1.84	17.6	15	15.8	32.4	85.1	52 U
4	Group 1	1.84	17.6	16	15.8	32.4	90.9	20 U
5		1.84	17.6	14		32.4		26 S[c]
6		1.84	17.6	15		32.4		33 S
7		1.84	17.6	16		32.4		13 S
8	2.72	1.76	20.0	12	18.0	35.2	60.0	62 U
9		1.76	20.0	13	18.0	35.2	65.0	50 U
10		1.76	20.0	14	18.0	35.2	70.0	43 U
11		1.76	20.0	15	18.0	35.2	75.0	36 U
12		1.76	20.0	16	18.0	35.2	80.0	29 U
13	Group 2	1.76	20.0	17	18.0	35.2	85.0	13 U
14		1.76	20.0	12		35.2		4 S
15		1.76	20.0	13		35.2		5 S
16		1.76	20.0	14		35.2		7 S
17		1.76	20.0	15		35.2		9 S
18		1.76	20.0	16		35.2		15 S
19		1.76	20.0	17		35.2		10 S
20	2.72	1.68	22.8	12	20.5	38.3	52.6	40 U
21		1.68	22.8	13	20.5	38.3	57.0	34 U
22		1.68	22.8	14	20.5	38.3	61.4	30 U
23		1.68	22.8	15	20.5	38.3	65.7	28 U
24		1.68	22.8	16	20.5	38.3	70.1	24 U
25		1.68	22.8	17	20.5	38.3	74.5	18 U
26	Group 3	1.68	22.8	20	20.5	38.3	87.6	3 U
27		1.68	22.8	12		38.3		2.5 S
28		1.68	22.8	13		38.3		3.5 S
29		1.68	22.8	14		38.3		4 S
30		1.68	22.8	15		38.3		5 S
31		1.68	22.8	16		38.3		6.5 S
32		1.68	22.8	20		38.3		4 S
33	2.72	1.60	25.8	12	23.2	41.2	46.5	28 U
34		1.60	25.8	13	23.2	41.2	50.4	24 U
35		1.60	25.8	14	23.2	41.2	54.2	22 U
36		1.60	25.8	15	23.2	41.2	58.1	20 U
37		1.60	25.8	16	23.2	41.2	62.0	18 U
38		1.60	25.8	18	23.2	41.2	69.8	11 U
39	Group 4	1.60	25.8	20	23.2	41.2	77.5	8 U
40		1.60	25.8	12		41.2		2 S
41		1.60	25.8	13		41.2		2.5 S
42		1.60	25.8	14		41.2		3 S
43		1.60	25.8	15		41.2		3.5 S
44		1.60	25.8	16		41.2		4 S
45		1.60	25.8	20		41.2		5 S

[a] Future maximum water content estimated at 90% of water capacity.

[b] U = unsoaked strength. [c] S = strength after soaking. Basic data from "Stabilization of Materials by Compaction," by Willard J. Turnbull and Charles R. Foster, Transactions, ASCE, Vol. 123, 1958, Fig. 2. Material : Weathered Loess near Vicksburg, Miss.

the exact conditions of testing are unknown, it would seem that the soaked specimen may have lost water before soaking, a condition that could have produced a higher saturation and, therefore, an additional loss in strength. Also it has been shown (38) that specimens during the period of soaking do not absorb equal percentages of water near the surface and in the interior. This also could cause discrepancies in strength. It would, therefore, seem that one or both of the causes might be responsible for the discrepancy.

TABLE 23.—RELATIONSHIP OF PORE SPACE AND SATURATION OF OMAHA RUNWAYS[a,b]

Density, in grams per cubic centimeter		Weight percent water		Pore space	
Apparent (1)	Bulk (2)	Capacity (3)	Content (4)	Total percentage (5)	Saturation percentage (6)
		Subbase			
2.62	1.85+	15.9-	14.4	29.4-	90.5
2.62	1.78	18.0	11.8-	32.0	98.7
2.62	1.75	19.0	16.2	33.2	85.2-
2.62	1.72	20.0	19.7	34.4	98.7
2.62	1.69	21.0	20.9	35.5	99.5+
2.62	1.69	21.0	19.6	35.5	93.2
2.62	1.69	21.0	20.2	35.5	96.1
2.62	1.68	21.3	20.5	35.8	96.2
2.62	1.68	21.3	21.0	35.8	98.5
2.62	1.66-	22.0+	21.5+	36.5+	97.6
Avg.	1.72	20.0	19.6	34.4	95.4
		Subbase			
2.63	1.85+	16.0-	15.0	29.6-	93.6
2.63	1.83	16.6	14.4-	30.4	86.6-
2.63	1.82	16.9	16.0	30.8	94.6
2.63	1.81	17.2	16.2	31.1	94.1
2.63	1.80	17.5	17.4	31.5	99.5+
2.63	1.78	18.1	17.3	32.3	95.5
2.63	1.78	18.1	17.6	32.3	97.2
2.63	1.78	18.1	17.5	32.3	96.6
2.63	1.78	18.1	17.5	32.3	96.6
2.63	1.77	18.4	17.8	32.6	96.7
2.63	1.71-	20.5+	19.7+	35.0+	96.0
Avg.	1.74	17.8	16.9	31.8	95.2

[a] Maximum, ; minimum, -. Basic data from "The Behavior of Densified Soil-Water Mixtures Under Very Adverse Conditions," by W. H. Campen and J. R. Smith, Proceedings, Highway Research Bd., Vol. 23, p. 450. Note. Columns 3, 5, and 6 computed for this paper by Ira B. Mullis.

[b] After Flood of 1943

9. In contrast, Nos. 13 and 19 have nearly identical values, a condition similar to that in a number of cases.

10. Concerning the great loss in strength of the soaked counterpart of specimen No. 33, summarized under G, the following comment is offered: as group 4 has the highest percentage of pore space of any other group and lowest

molding or starting-out-content of water—the lowest saturation percent—it seems logical that it should be expected to absorb the greatest volume of water; therefore, it should be expected to lose more strength on soaking than any other specimen.

11. The higher the percentage of pore space and the lower the percentage of saturation of a specimen of earth at the time it comes in contact with water, the greater its consequent percentage of absorption, swell and loss of strength.

12. The lower the percentage of pore space and the higher the percentage of saturation of a specimen of earth at the time it comes in contact with water, the less its consequent percentage of absorption, swell, and loss in strength.

Airport Runways Withstand a Flood.—In the year 1943, the runways of the Omaha Airport, Omaha, Nebraska became submerged from 3 ft to 7 ft during a period of 10 days to 17 days. The roadbed of the runways consists of a 12 in. soil compacted subbase with a PI of 18% to 29% laid on a Missouri River silty clay; a 6 in. compacted sand-clay-gravel base, and a 2 in. to 3 in. bituminous surface.

TABLE 24.—AVERAGE PORE SPACE, WATER CONTENT AND SATURATION OF RUNWAYS, OMAHA AIRPORT AFTER MISSOURI RIVER FLOOD OF 1943.

	Weight percent water			Pore space	
	Capacity (1)	Content (actual) (2)	Content (future)[a] (3)	Total percentage (4)	Saturation percentage (5)
			Subbase—No. tests, 75		
Maximum,	23.4	22.3	21.8	37.6	99.0
Minimum,	14.6	13.2	13.6	27.6	75.6
Average,	19.3	17.7	17.7	33.7	92.6
			Base—No. tests, 17		
Maximum,	8.5	7.0	7.6	18.2	95.9
Minimum,	5.8	4.9	5.0	13.1	79.4
Average,	7.1	5.8	5.6	15.0	87.3

[a] The estimated maximum future water content to be expected is 90% of the water capacity of the pore spaces. Values in the table are from "The Behavior of Densified Soil-Water Mixtures Under Very Adverse Conditions," by W. H. Campen and J. R. Smith, Proceedings, Highway Research Board, Natl. Research Council, Vol. 23, 1943, p. 450.

From test data obtained by Campen and his associates immediately after the subsidence of the flood, characteristic conditions are shown in Table 23 and a summary in Table 24. This research is fortunate in having access to test data on runways after they have been subjected to the flood conditions previously described. Attention is called to the limited pore space and its saturation by the subbase and the base also. In reviewing these values, one should remember that though pore suction aided in the absorption of water, the hydraulic head must have contributed most. Another important feature of Table 24 is that shown in column 5 that indicates that a grand average of only 92.6% of the pore space became filled, even under the very adverse conditions imposed by the flood. Attention is called also to the close approach of estimated and actual values shown in columns 2 and 3. Of the bases, similar comments seem applicable, though it is apparent that as usual the excellent bases seem to offer more resistance to the penetration of water than do the clays. Only one or two specimens showed a saturation of 100%.

TABLE 25.—EFFECT OF PERCENT OF PORE SPACE AND SATURATION ON SHRINKAGE

Density, in grams per cubic centimeter		Weight percent water			Pore space		Volumetric percent shrinkage (8)
Apparent (1)	Bulk (2)	Capacity (3)	Content (original) (4)	Content (future)[a] (5)	Total percentage (6)	Saturation percentage (7)	
2.65[b]	1.70	21.0	10	18.9	35.8	47.6	5.7
2.65	1.70	21.0	12	18.9	35.8	57.6	4.4
2.65	1.70	21.0	14	18.9	35.8	66.6	2.6
2.65	1.70	21.0	16	18.9	35.8	76.1	1.6
2.65	1.70	21.0	18	18.9	35.8	85.5	0.7
2.65[b]	1.80	17.8	10	16.0	32.0	56.1	5.6
2.65	1.80	17.8	12	16.0	32.0	67.4	4.1
2.65	1.80	17.8	14	16.0	32.0	78.5	2.5
2.65	1.80	17.8	16	16.0	32.0	89.7	1.4
2.65	1.80	17.8	18	16.0	32.0	99.0	0.0
2.65[b]	1.85	16.3	10	14.7	30.0	61.3	5.5
2.65	1.85	16.3	12	14.7	30.0	73.6	3.6
2.65	1.85	16.3	14	14.7	30.0	85.7	1.5
2.65	1.85	16.3	16	14.7	30.0	98.1	0.8
2.65	1.85	16.3	18	14.7	30.0	?	0.0
2.65[b]	1.60	24.6	10	22.2	39.5	40.6	5.5
2.65	1.60	24.6	12	22.2	39.5	48.7	3.6
2.65	1.60	24.6	14	22.2	39.5	56.9	2.6
2.65	1.60	24.6	16	22.2	39.5	65.0	1.8
2.65	1.60	24.6	18	22.2	39.5	73.1	1.2
2.65[b]	1.51	28.4	10	25.6	43.0	35.2	1.0[c]
2.65	1.51	28.4	12	25.6	43.0	42.2	3.2
2.65	1.51	28.4	14	25.6	43.0	49.2	2.6
2.65	1.51	28.4	16	25.6	43.0	56.3	1.6
2.65	1.51	28.4	18	25.6	43.0	63.4	---

[a] Estimated future maximum saturation 90% of water capacity.

[b] Assumed value. [c] Value believed to be the lowest in the group due to large pore space and low saturation.

Note.—Bulk density, percent moisture when compacted and percent of shrinkage from "The Results of Tests to Determine the Expansive Properties of Soils," by Harold Allen and A. W. Johnson, Proceedings, Highway Research Bd., Natl. Research Council, 1936, p. 220. Numerical values in the above table taken from the discussion by Delbert L. Lacey of "Design of Roadbeds," by Ira B. Mullis, Transactions, ASCE, Vol. 111, 1946, Table 8.

States and Properties of Clay.—Nature has decreed that solid rocks can be decomposed into clay mostly by the processes of drying and wetting and by heating and cooling. Therefore, rocks hidden deep in the earth where change is the slightest, retain their strength the longest and those exposed to rapid change at the surface are most rapidly changed to clay-like matter. As clays, unlike rocks, have been weathered to the end-point, they undergo little chemical change; however, under a favorable environment they undergo enormous physical change. With varying amounts of water, clay may range from a slurry to a solid with little water. As a slurry, it can sustain loads only by its buoyant force; but when its pore space is reduced to the minimum, and almost filled with water, its toughness and strength are at the maximum. In the powdered state, it is extremely weak; but when the pore space is small and filled with air, it is a strong brittle solid.

Like solid rock, a particle of clay can swell or shrink very little; but when it contains a fluid such as air or water, it shrinks or swells in proportion to the loss or gain of fluid content. Consequently, no change in volume can occur if the fluid content remains constant regardless of the volume of pore space. However, if the body of clay is to be made and kept strong, pore space must be correspondingly small and its percentage of saturation kept constant.

Table 25 shows that when pore space and saturation are high and drying occurs, shrinkage is greatest. Due to variation to be expected in all tests, some variation is shown, but the trend can hardly be questioned.

Fig. 5 shows a loose gumbo that absorbs large quantities of water and undergoes considerable volume change during wetting and drying. When puddled, shrinkage usually occurs in the form of fissures. Wherever gumbo clay is found, its strength varies enormously with water content, but this variation in strength is often controlled by the use of consolidated sand-clay mixtures to such an extent that these roadbeds carry great loads. Fig. 6 shows a sand-clay road surface on a gumbo roadbed characterized by the lack of loose material. Due to the strong bond of gumbo to the sand or gravel, these surfaces resist dry weather better than do less adhesive clays.

Does Capillary Water Accumulate Under Pavements ?—Additional support is given to premises 17, 18, and 19 by Oscar Edward Meinzer (40) when he states that if the capillary fringe is near enough to the ground surface to lose water by evaporation or by the absorption of roots of plants, there is a continuous movement of water from the water table upward through the capillary interstices.

The Encyclopedia Britannica (41), says the cohesion theory of the assent of sap requires only that one membrane is evaporating into dry air, another membrane is in contact with a water supply, and the two connected by a continuous water thread under such conditions that it can support a tensile stress.

In view of the experimental evidence that evaporation is an essential to the movement of capillary water and with no experimental evidence to the contrary, it seems that capillary action cannot produce an accumulation of water at the discharge end of a wick-like material of any kind including earth.

Frost Action in Roadbeds.—Under the side heading, "Frost Action," frost action is discussed largely on the basis of experience gained during this research, but it will now be confined to the research work of others.

Frost Effects on Solid and Unconsolidated Bodies.—Earnest de K. Leffingwell (42), after several years of exploration and research in northern Alaska, notes that, of course, ground ice is not to be expected in hard rock, but in what would be considered as unconsolidated deposits, if they had not already been solidified by ice. Of the unconsolidated deposits, some were more favorable to frost action than others. Coarse sand and gravel deposits seem

FIG. 5.—A POROUS GUMBO CLAY THAT ABSORBS MUCH WATER

FIG. 6.—A CONSOLIDATED SAND-CLAY SURFACE ON A GUMBO ROADBED

to have much less ice than clay and silt. The most favorable material is muck.

Small Pores and Cracks Least Resistant to Frost.—Stephen Taber (43), another research worker on frost action in Alaska, states that ice crystals can best develop in relatively permeable, fine-grain rocks having numerous small pore spaces. Normally, fine-grain rocks that are highly impermeable, and coarse-texture rocks with relatively large open spaces, are most resistant to frost action. Due to lack of water, frost effects are practically absent on bare rocky ridges and cliffs; but at the foot of steep slopes where water accumulates, frost destruction is relatively rapid. The ice content of silts may be as much as 80% by volume and the average is probably in excess of 50%. Such silts on thawing turn to semi-liquid mud that flows where opportunity exists for escape.

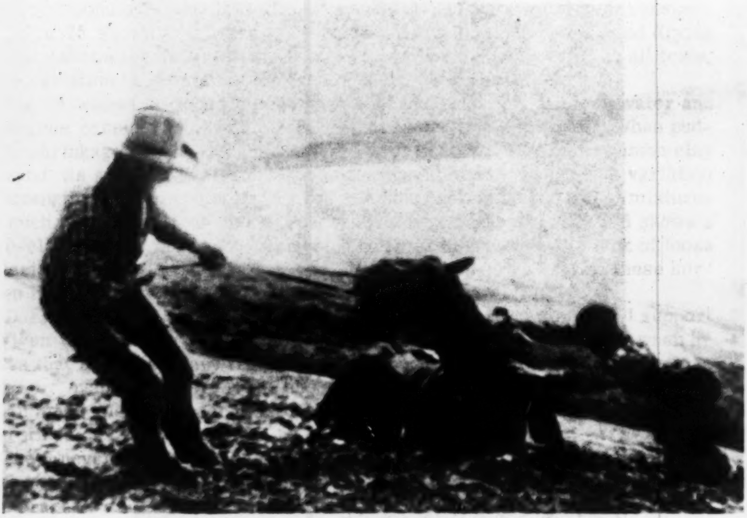


FIG. 7.—A HORSE ENGULFED IN A QUAGMIRE AT THE FOOT OF A SNOW-COVERED SLOPE

In support of the statement by Taber concerning the relatively rapid destruction at the foot of slopes, Fig. 7 is shown. The horse engulfed in the quagmire located at the foot of a slope illustrates the result of frequent freezes and thaws. The dark belt near the top of the photograph shows ground on which the snow has completely thawed. Below the belt, snow is available for additional thaw water. South-facing slopes, because of their more frequent exposures to the sun usually supply more thaw water during alternate freezes than do north-facing slopes that remain frozen until evaporation lessens the runoff. Of course, many repetitions of freezes and thaws, enlarge pore space and water refills it until the area becomes a treacherous quagmire. Where cattle have access to these quagmires they oftentimes become engulfed.

Glacial Quarrying in Jointed Rocks.—According to Richard F. Flint (44) well-consolidated clay-rich till, virtually free of joints resists quarrying almost perfectly and, where glacially eroded, shows the effects of abrasion only. Once the glacier has removed the weathered mantle and has worked down through the open-jointed bedrock beneath, the quarrying becomes sharply reduced and continues to diminish as joints become tighter with depth. Fig. 8, after F. E. Matthes, shows how frost-welding, quarrying or plucking-out in the more porous or jointed rocks are changed into step-like forms during glaciation on slopes. The steps are shown to be controlled by the unequal distribution of joints and other planes of weakness that cut the bedrock, though other factors may affect the production of such steps. The dashed line indicates the valley profile before glaciation; the dotted line indicates the profile during an early phase of the glaciation.

Although rocks of limited pore space do resist frost action to a remarkable degree, conditions occur that cause serious rupture. As rocks at considerable depths are likely to be highly saturated with water, many quarries supplying dimension stone in very cold regions are forced to discontinue quarrying to avoid destructive frost rupture before adequate evaporation can occur.

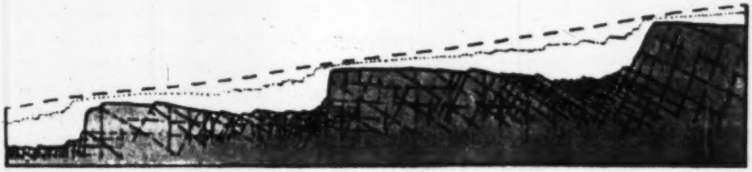


FIG. 8.—EFFECT OF ROCK JOINTS ON GLACIAL QUARRYING

Following this analogy, a consolidated body of clay with pore space more than 90% saturated must also rupture under a temperature that freezes the entire water content. However, if a roadbed is carefully constructed, experience teaches that 100% saturation is not likely to attend construction. However, when roadbed construction is carelessly done so that pore spaces contain large quantities of air, vibration from traffic, changes in barometric pressure and precipitation are likely to produce a saturation that is sufficient to cause considerable damage, especially during a deep freeze immediately following considerable precipitation. On the other hand, when pore space is reduced to small percentages and saturation is ample, additional saturation seems to be improbable as is indicated in Table 22.

Conclusions.—As the bedrock foundation of the paper rests on premises 1 through 21 near the beginning and in the section entitled "Reduction of Pore Space Near Saturation Increases Strength," items 1 through 12, are regarded as appropriate conclusions.

Grateful appreciation is expressed to those who have provided much basic data and also to those who have contributed to the discussion. Without these, the paper, in its present form, would have been impossible. Gratitude is also expressed to L. Polk Denmark for his preparation of Fig. 4.

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TRANSACTIONS

Paper No. 3259

DEVELOPING ALTERNATIVE URBAN TRANSPORTATION SYSTEMS

By William R. McConochie,[1] F. ASCE

With Discussion by Messrs. Kenneth M. Hoover; S. D. Forsythe;
Nathan Cherniack; and William R. McConochie

SYNOPSIS

Principles of urban transportation applying to most large cities were revealed by a comprehensive study of highway and transit needs of Metropolitan Washington. This paper, based primarily on civil engineering studies, discusses costs, advantages, and disadvantages of auto-dominant, all-bus, all-rail, and recommended plans for Washington, D.C.

OBJECTIVES OF THE STUDY

A Transportation Plan for the National Capital Region was presented to the President in July, 1959, and promptly forwarded to the Congress and to various interested government agencies. The report was the culmination of a comprehensive transportation survey of the Washington region directed by the National Capital Planning Commission and the National Capital Regional Planning Council. A great many organizations and individuals participated in the work, including ten consultants, most of the latter supported by sizeable staffs. This paper is based on a portion of the civil engineering consultant's phase of the survey.

Note.—Published essentially as printed here, in March, 1960, in the Journal of the Highway Division, as Proceedings Paper 2417. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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The findings of the study, while having specific reference to Washington, are so fundamental that they should prove helpful in formulating transportation policies in other major cities, but a warning seems in order at this point. Most large cities have striking similarities, particularly in regard to their transportation problems. Washington is unique in many ways, however, the most significant factor affecting transportation being the physical character of its central business and government district. While some cities with greater population have a core less than 1 sq mile in area, Sector Zero in Washington—essentially the area encompassed by the proposed Inner Loop Freeway—has an area of 6½ sq miles. The streets in this area are wide, the heights of the buildings are kept within rigid limits, and many acres are devoted to parks and extensive grounds of public buildings.

These characteristics have a profound influence on the physical nature of any practical plans for both highways and transit systems. The findings of the Washington study cannot safely be applied to any other city, therefore, without first evaluating the difference between the two cities, particularly in regard to their downtown areas.

HIGHLIGHTS OF THE WASHINGTON PLAN

The recommended transportation plan for the National Capital Region evolved slowly from the findings of numerous interlocked studies. One of the techniques used was to evaluate various hypothetical transportation systems without immediate concern for their practicality. The results of these preliminary studies were of immeasurable value in guiding subsequent steps toward a sound and realistic plan.

The study included not only the movement of people in automobiles on modern highways and in public transit vehicles on freeways and parkways, but also the parking needs of automobiles and the highway requirements of trucks. The operation of transit vehicles in local service was not included in the study. The target date was taken as that year when the population of the study area reaches 3,000,000. This is predicted to be about 1980. Population in 1955 was about 1,900,000.

Schemes Studied.—Four basic schemes were studied:

Plan I—the auto-dominant system—would consist of an extensive network of freeways, parkways, and arterial streets together with large-capacity downtown parking facilities on the assumption that a steadily increasing proportion of people will use private automobiles. Public transportation would consist of buses sharing the streets and highways with other automotive traffic as at present.

Plan II—the express bus system—would consist of high-speed buses making few stops and traveling on freeways and parkways—or on their own grade-separated roadways if need be—and maintaining a high standard of transit service. Local buses would supplement the express bus service. The need for highways and downtown parking facilities would be less than under the auto-dominant system.

Plan III—the rail or other train-type transit system—would be on grade-separated ways located either in the center malls of radial freeways or on other exclusive rights-of-way, with trains providing fast service. As with the express bus system, local bus service would be provided. The need for highways and downtown parking facilities would be less than with the auto-dominant system.

Plan IV—the information obtained from these three basic study plans was used to formulate a recommended transportation plan to facilitate the optimum movement of people and goods in the National Capital Region.

The schemes studied are shown in Figs. 1 and 2.

While the recommended plan actually grew out of the other three plans, the recommended plan will be described first. It can then be used as a yardstick for measuring the others.

PLAN IV—RECOMMENDED HIGHWAY-TRANSIT SYSTEM

The recommended transportation system to serve the National Capital Region would be composed of a $2,000,000,000 network of freeways, major streets, and parking facilities together with a $500,000,000 system of rail rapid transit and express bus facilities. Express buses would be used in the early years on the routes ultimately to be served by rail rapid transit.

The proposed transit service would consist partially of rail rapid transit trains running on separate rights-of-way. It would also include express buses operating in the general traffic stream on freeways and parkways. A supporting system of local bus routes would be provided as an integral part of the service, but the capital investment in this portion of the system is not in the estimated cost. The local buses would serve as feeders to the network of rail and bus rapid transit lines and as distributors of express traffic in the central business district. They would also provide transportation in areas where express service would not be justified, both in the District and in the suburbs.

Throughout this paper the terms "express" and "rapid transit" are used interchangeably to indicate service by buses on freeways and parkways as well as by rail vehicles on private rights-of-way.

Coordination Between Highways and Transit.—Careful coordination between highway and transit systems promises substantial economies over the total capital cost of separate and competing systems. Rail facilities are planned to share rights-of-way with highways in some instances, and freeways will be utilized for operation of express buses where appropriate. Further savings have been anticipated by providing public transportation so attractive that the use of transit will somewhat reduce the need for highways. This is particularly important in providing good transportation to and through areas where the costs of rights-of-way for additional freeways would be excessive.

Despite the extensive rapid transit system contemplated, however, an elaborate network of freeways and parkways will be necessary by 1980. The transit system, for example, could not serve people who need their vehicles during the day for trips in which the automobile is essential because of the nature of the journey. This group alone accounts for approximately 30% of all the estimated automobile trips that would occur under the assumptions of the auto-dominant system. Another large group that could not be drawn to transit consists of people moving between points within the survey area and points outside—classified as external-internal trips—and those making through trips entirely across the survey area from external points to external points. Finally, the vehicular trips indicated for the highway system include the movements of commercial vehicles translated into an equivalent number of automobiles.

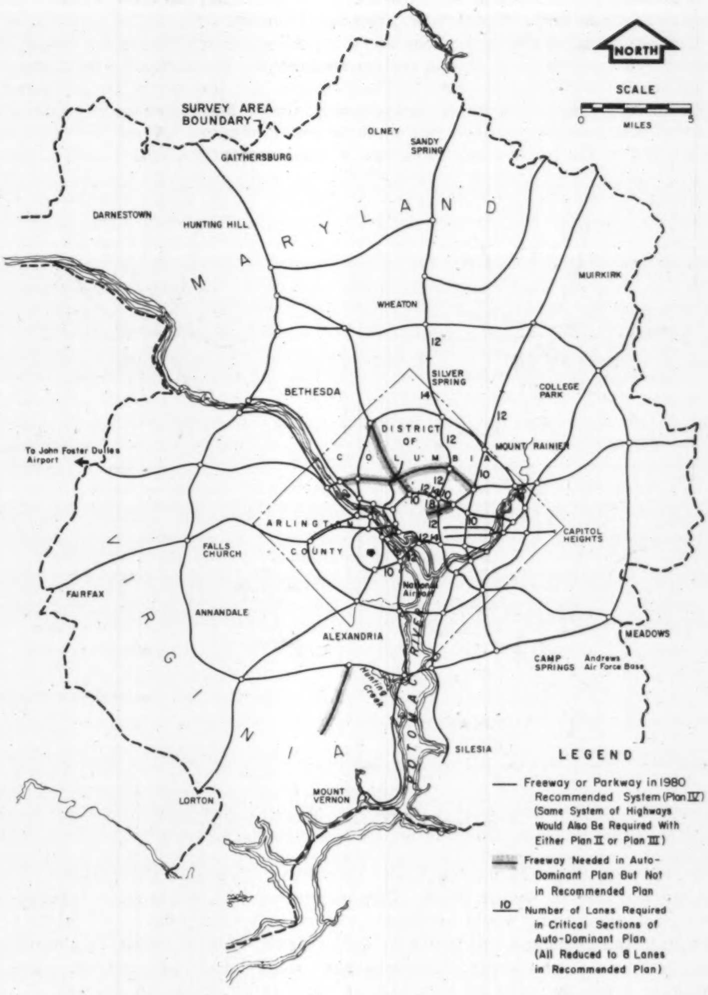
NORTH
SCALE
0
MILES
5
SURVEY AREA BOUNDARY
OLNEY
SANDY SPRING
GAITHERSBURG
DARNESTOWN
HUNTING HILL
MARYLAND
MUIRKIRK
WHEATON
12
SILVER SPRING
14
BETHESDA
COLLEGE PARK
12
DISTRICT OF COLUMBIA
12
MOUNT RAINIER
10
To John Foster Dulles Airport
12
12/10
10
18
12
10
12/14
ARLINGTON COUNTY
CAPITOL HEIGHTS
FALLS CHURCH
10
National Airport
FAIRFAX
ANNANDALE
ALEXANDRIA
MEADOWS
VIRGINIA
POTOMAC RIVER
CAMP SPRINGS
Andrews Air Force Base
Hunting Creek
SILESIA
MOUNT VERNON
LORTON
LEGEND
Freeway or Parkway in 1980 Recommended System (Plan IV) (Same System of Highways Would Also Be Required With Either Plan II or Plan III)
Freeway Needed in Auto-Dominant Plan But Not in Recommended Plan
10 Number of Lanes Required in Critical Sections of Auto-Dominant Plan (All Reduced to 8 Lanes in Recommended Plan)

FIG. 1.—AUTO-DOMINANT AND RECOMMENDED FREEWAY SYSTEMS

Trucks are estimated to represent 16% of the total vehicular trips in the area during a normal 24-hr weekday in 1980 under the auto-dominant estimates. This is the equivalent of 27% of total 24-hr traffic when the effect on the traffic stream of these large vehicles is taken into account.

Notwithstanding the large blocks of motor-vehicle-imperative trips, the transit system was found by the traffic engineering consultant to be attractive to substantial numbers of people. The greatest service would be rendered to those making trips to or from the central business and government district or to focal points in adjoining areas, such as the Pentagon. Of all the estimated 1980 trips on the rapid transit system, approximately 6% would consist of intraline trips which would not reach the downtown area and 61% would have origins or destinations in Sector Zero. The remaining 33% would pass through this area, but many trips would terminate just outside its borders. Of all the trips drawn to the proposed rail and bus rapid transit system, about 70% would represent people who would otherwise have used local bus service. Only 30% would be made by those who would have used automobiles as drivers or passengers.

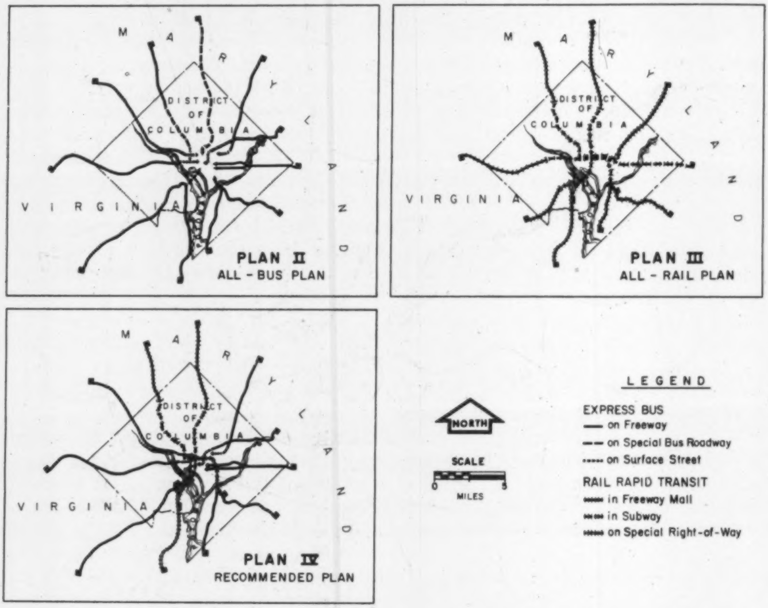
MARYLAND
DISTRICT OF COLUMBIA
VIRGINIA
PLAN II
ALL - BUS PLAN
PLAN III
ALL - RAIL PLAN
PLAN IV
RECOMMENDED PLAN
NORTH
SCALE
0
5
MILES
LEGEND
EXPRESS BUS
— on Freeway
– – on Special Bus Roadway
····· on Surface Street
RAIL RAPID TRANSIT
in Freeway Mall
in Subway
on Special Right-of-Way

FIG. 2.—ALTERNATE TRANSIT PLANS

Thus, it can readily be seen that a good rapid transit system can complement a network of urban freeways, but it cannot provide an adequate substitute for modern highway facilities.

Recommended Highway System.—Even an over-designed system of rapid transit facilities would not drastically reduce the need for freeways. With the recommended transit system, the highway needs for 1980 are estimated at 67 miles of additional freeway and parkway routes over those now existing or planned, for a total of 326 miles. Many existing or proposed highways should be made wider than previously contemplated. The recommended system is illustrated in Fig. 1.

Improvements to Arterials.—The proposed network of freeways and parkways will have to be supplemented by a comprehensive system of expressways, park roads, major streets, and other arterial thoroughfares, principally at surface grade. They will consist mainly of existing adequate thoroughfares, widened pavements within existing rights-of-way, extensions to connect interrupted streets, and grade separations at complex intersections. These streets will act as feeders and distributors for the network of limited access highways. They will also carry the heavy volumes of traffic making short trips or trips in corridors that would not support more elaborate facilities.

Recommended Rail Routes.—Two basic routes would be served by rail rapid transit under the proposed plan. One would extend in a north-south direction from near Wheaton to south of Alexandria, a distance of 20 miles. The other would reach from the northwest area north of Bethesda to a point beyond the Anacostia River, southeast of the central business district. The length of this rail line would be about 14 miles, for a total of 34 miles. Of this, about 14 miles would be in subway, one-half within the out-size central business and government district.

The north-south route would start at a point near the Interstate Circumferential in Montgomery County, where a bus transfer station would be provided together with adequate parking facilities. A double-track rapid-transit railroad would be built, it is proposed, in the center mall of that portion of the north central freeway extending generally from the vicinity of Wheaton to a point within the District. The exact point would be determined by the alignment of a proposed freeway. The line would enter a subway in the roadbed of New Hampshire Avenue and continue southwesterly to 13th Street N.W. With various minor jogs, it would continue in subway through the central business district, passing over the other proposed subway route in E Street N. W.

The line would emerge from the subway just beyond Washington Channel and cross the Potomac River on a new bridge. After serving the Pentagon, it would continue on fill, principally along railroad properties, past the Washington National Airport, and through Alexandria.

A terminal would be built south of Hunting Creek near Jefferson Manor. At this point a bus-to-rail transfer facility would be installed as well as a parking lot. In all cases, such outer terminals would also have adequate facilities for delivering or meeting passengers with cars which would be parked only momentarily.

Another rail rapid transit facility would start from a terminal in the vicinity of Pooks Hill, north of Bethesda, Md. A freeway route in this corridor is proposed, and the rail line would occupy the center mall from the outer terminal to a point inside the District. As with the northerly portal on the north-south subway route, the exact point would depend on the freeway location. From that point, the rapid transit facility would consist of a subway in the bed of existing streets for most of the remaining distance to the southeast terminal.

Construction.—Stations would be made attractive to the patrons by incorporating all of the architectural features characterizing the most advanced design.

Walls and other surfaces which the public could touch would be faced with structural glass, tile, stainless steel, or other mar-proof and easily cleaned materials. Full use would be made of acoustical materials to keep the noise level at a minimum. Fluorescent lighting would be applied generously in all parts of the stations and in their vicinity. Escalators would be provided at many places, their use now being practical even where the device is partially exposed to the elements. In the downtown area, entrances would be opened, it is proposed, between subway stations and basement levels of stores and office buildings.

Subways would be built, for the most part, by use of cut-and-cover methods. This would keep the vertical distance between sidewalk level and platform level to a minimum and would be the most practical design for conditions in Washington. Street traffic would be interrupted only temporarily with this method of construction, since the street would be restored by decking over the excavation after the initial stage had been completed.

Passenger Equipment.—Several new types of rapid transit cars have been developed in recent years. It is difficult to select between them for qualities of comfort, appearance, and ease of maintenance. It is likely, however, that an articulated car consisting of three units permanently coupled on four trucks will prove to be the most appropriate for the Washington operation. This design permits lightweight construction with economy in use of materials and yet sufficient flexibility to permit trains to vary in length from 150 ft to 450 ft. Station platforms would be 500 ft long. The interior of the cars would be finished in modern synthetic materials as well as in corrosion-resistant metals for ease of cleaning. Lighting would be kept at high intensity, and all equipment would be fully air-conditioned to assure a supply of clean, fresh air at comfortable temperatures in all seasons.

The trucks of the articulated units could have rubber-tired wheels—as in the case of the latest Paris trains which are now (1960) in successful operation after a development period of many years. On the other hand, they might have steel wheels operating on steel rails in the conventional manner, but perfected to be very quiet, similar to the equipment used in the new subway in Stockholm.

Recommended Express Bus Routes.—In addition to two main rail rapid transit lines, comprised of four radial routes, eight distinct express bus routes are proposed, serving all remaining segments of the metropolitan area. The bus routes, similar to the rail routes, would have their own systems of bus feeders. In some cases, a physical transfer of passengers at outer or intermediate stations would be involved. In other instances, however, express buses might be through-routed between the freeways and the surface streets on the suburban portions of their routes.

On all routes, outer terminals would consist of adequate facilities for bus-to-bus transfers, as required, together with extensive parking space. In many cases the terminals would be built only when the territory beyond had been developed, temporary facilities serving in the interim. Bus stations would be provided along the freeways at approximately 1-mile intervals. The spacing would depend somewhat on the location of intersecting major streets which would serve feeder buses as well as persons walking, driving or being driven to the stations.

The largest buses practical for urban operation would be used in the express service—probably of about 52-seat size. Feeder service to the outer terminals might be rendered in some instances with smaller buses appropriate to route characteristics.

Two or more types of service would be operated between the outer terminals and the points of leaving the freeways near the downtown area. During peak periods on all routes, and even during midday on the heavier lines, some buses would run non-stop from the suburban terminals to downtown. Other buses would stop at every station or every second station.

At appropriate points in the vicinity of the Inner Loop Freeway, the express buses would leave the radial freeways and continue their trips on the surface streets within the central area. On most express bus routes, the peak hour flow of buses would exceed the number that could be operated efficiently on a single surface street, especially since most downtown streets will also be carrying local buses. Each express route, therefore, would have to fan out into two or more downtown distributor routes. This would broaden the distribution within the area, but some patrons would still find it necessary to transfer to other bus lines or to the proposed subways to reach their ultimate destinations. Washington's broad downtown streets and low density of development makes this scheme for distribution by bus feasible. The plan might not be practical, however, in a city with a more intensely developed business district.

The recommended transportation system is designed to be adequate to serve a population of 3,000,000 in the National Capital Region. The number of buses on the various routes—from about 50 to 250 in each direction in the maximum hour—would be such that they could be absorbed into the general traffic stream. As the population of the area approached 3,000,000, however, special ramps for the exclusive use of buses might have to be built in the vicinity of the Inner Loop Freeway. The cost of such ramps is not included in the estimates. It might also be necessary to restrict the volumes of traffic on certain sections of the freeway and parkways system as the target date was approached or as segments of the incomplete system were placed in operation. These steps would be taken not only for the benefit of bus patrons, but in order to keep all traffic moving. As population grew beyond 3,000,000, special roadways for buses might have to be built, or the rail system might have to be expanded to supplant bus routes.

Local Bus System.—A system of local bus routes would operate on the improved major streets to carry passengers to and from the stations of the rapid transit system. Local routes would also provide service for people making trips independent of that system as at present. It is presumed that existing transit company operations would be coordinated with the proposed rapid transit system.

Conclusions.—In order to keep the requirements for freeways within reasonable bounds, it was necessary to assign rush-hour traffic to the major streets, as proposed to be improved, to approximately 80% of their theoretical capacity. Thus, the capacity added to the total system by building $2,000,000,000 worth of limited-access highways and other facilities would do little more than absorb the year-by-year increases in traffic. Rush-hour congestion on surface streets would not be greatly lessened.

Another disconcerting finding of the traffic engineering consultant is that—despite an investment of $500,000,000 in rapid transit facilities—the percentage of total passenger trips in the metropolitan area by public transportation would drop from 20.4% in 1955 to 18.4% in 1980.

Questions arise immediately as to whether or not it would be more effective in solving the area's transportation problems to expand the highway system and spend less for a transit system, or on the other hand, if it wouldn't be better to de-emphasize highways and build even more elaborate transit facilities. Such questions were answered by the procedure followed in the survey. The auto-dominant and transit-dominant plans were prepared first, however, and then the recommended plan was developed from the best features of the more extreme plans.

PLAN I—AUTO-DOMINANT PLAN

The term Auto-Dominant Plan was applied to that system of freeways and parkways which would be adequate to serve a major portion of the travel needs of the metropolitan area, with transit playing a minor role. Public transportation would consist of local buses operating on the surface streets, giving service of the type and relative extent provided in 1955. The patrons of this system would be those whose trips were "transit-oriented"—essentially dependent on transit for their journeys about the region. Under these assumptions, the traffic consultant estimated that approximately 14% of all person-trips in the area in 1980 would be made by public transportation, while 86% would be made by private automobile. This would be a one-third drop in relative use of public transportation between 1955 and 1980.

In estimating the requirements for highway capacity under the auto-dominant plan, the needs of commercial vehicles were recognized. The average truck was taken as the equivalent of approximately two passenger automobiles in computing the required rush-hour capacity.

Composition of Network.—In order to permit civil engineering estimates of the number of lanes required in each section of this more extensive road network, the traffic consultant assigned the predicted peak hour two-way weekday vehicular traffic in 1980 to the auto-dominant freeway system. All traffic assignments were first based on the assumption that every freeway in the system would have ample capacity for relatively fast, fluid movement at all times.

The auto-dominant plan of highways as finally developed by this method would consist of a network of 344 miles of freeways and parkways within the study area. This is only 86 miles more of this type of highway than currently existing or programmed, but the lane-miles would be 45% greater. The system would consist essentially of 15 radial routes. These would be interconnected by a series of concentric circles consisting of the Inner Loop Freeway system, an Intermediate Loop, and a Circumferential lying at a distance from the White House of 7 miles to 11 miles.

Critical Sections.—The corridor of heaviest demand in the auto-dominant plan extends from the southwest leg of the Inner Loop Freeway along the previously proposed center leg of the Inner Loop Freeway paralleling 3rd Street S. W. and N. W. and thence northerly on an undefined alinement through the District to the Circumferential in the vicinity of Silver Spring. While the need for a highway through this corridor has been recognized, no definitive studies have been made on a freeway route northerly from the north leg of the Inner Loop Freeway. Twelve to eighteen lanes would be required to carry traffic in various sections of this route under the assumptions on which the auto-dominant plan was based. Other demands imposed by the assumptions of the auto-dominant plan which would be difficult, if not impossible, to meet would be a corridor from the east end of Highway (14th Street) Bridge via the southwest route of the Inner Loop Freeway to 11th Street S. E. In this section, as many as fourteen lanes would be required, whereas no more than eight can readily be provided or made to function. Moreover, the high proportion of traffic going to and from the central business district via this route, under the assumptions, would overload the streets serving the freeway ramps.

Estimated traffic volumes in this section would be even higher, furthermore, were it not for the assumption of a downstream four-lane bridge or tunnel across the Potomac River. This crossing would connect the Maryland and Virginia portions of the Intermediate Loop approximately at the south edge of the Washington National Airport. If this expensive connection ($53,000,000 for a tunnel) were to prove infeasible, the requirements along the south side of the Inner Loop Freeway would be increased by approximately two lanes under the auto-dominant system assumptions.

The remaining grossly overloaded section of the auto-dominant system is that portion of the Inner Loop Freeway comprising the north side of the Loop between approximately DuPont Circle and New York Avenue. While the studies made in connection with the preliminary designs for this freeway in 1955 proved that an eight-lane freeway was the maximum practical construction in this section, the estimates for the auto-dominant system showed a need in this corridor for as many as fourteen lanes. Even this excessive demand was kept down only by proposing two rather expensive sections of freeway not heretofore proposed in any official plan.

Estimated Cost.—The auto-dominant system of highways would consist of 2,032 lane-miles of limited access freeways and parkways, of 15% more than in the recommended system. The cost, at 1958 prices, including over $200,000,000 for street widening and other major highway construction, would be approximately $2,371,000 or 30% more than for highways in the recommended plan.

Parking Demand.—The number of automobiles to be parked in Sector Zero at the moment of peak accumulation under the assumptions of the auto-dominant plan would be approximately 1.6 times the number parked in that area during the survey of 1955. A great deal of curb parking would have to be eliminated in order to provide street capacity in the area for moving traffic. Furthermore, an estimated 22,700 existing spaces in off-street lots will probably disappear between now and 1980 due to the construction of new buildings, many of which are programmed to be built by the Federal Government. It is estimated, therefore, that 85,000 new off-street spaces would have to be provided to serve the needs of motorists under the auto-dominant plan. This is equivalent to about 40 garages as large as Chicago's Grant Park Underground Garage. The total cost is estimated at approximately $236,000,000, even assuming that the great demand for appropriate sites for garages would not inflate land values. For the recommended plan, 48,500 new parking spaces would have to be built in the downtown area, or 57% as many. Even this calls for the construction of one mammoth garage every year or equivalent, for the next 20 yr.

Conclusions.—There are serious engineering questions as to the practicality of the auto-dominant plan. Outer portions of the system could undoubtedly be made to work—at least until 1980. Key sections of the freeway system, however—particularly the Inner Loop Freeway and the proposed north central freeway—would require more lanes than could be built within limitations imposed by rights-of-way problems, and more than would be functionally operable. Furthermore, there is little likelihood that the streets in the central area could accomodate the number of vehicles which would be moving to and from the freeway ramps with the estimated volumes of traffic.

The capital cost of the auto-dominant plan would be only about 5% greater than that of the recommended total transportation plan. The land takings for highways and parking garages under the auto-dominant plan, however, would be so great as to cause immense indirect damages, particularly in and around the central core.

A transportation policy requiring the National Capital Region to be served almost exclusively by automobiles would be incompatible with the distribution of living, shopping and working places of the area's inhabitants as foreseen by the staff of the Commission and Council. If a regional population of 3,000,000 is to be served principally by automobiles, the central core must be relegated to a less important role, and radical decentralization of most of its functions must be accepted as the pattern of future growth.

PLAN II—ALL-BUS RAPID TRANSIT PLAN WITH RECOMMENDED SYSTEM OF HIGHWAYS

The recommended transit scheme is composed of both express bus routes on freeways and rail lines on insulated rights-of-way. This combination was selected, however, only after testing, first, a rapid transit system consisting solely of express bus routes on freeways and separate bus roadways; and second, a system composed exclusively of rail facilities.

Both the all-bus and the all-rail transit plans were initially considered in connection with a highway system that would be limited to the freeways, parkways and major streets now existing or definitely planned. This system of highways was found to be so inadequate for 1980 traffic, however, regardless of the best companion transit system that could be devised, that an enlarged highway network was developed. This highway system would be necessary in connection with either the all-bus or the all-rail transit plan. The highway system as thus evolved was finally joined with the proposed combination bus-rail transit system to form the recommended comprehensive transportation plan.

The rapid transit system involving the lowest investment for special transit facilities would be comprised of express bus routes on freeways built for general traffic, with separate lanes for buses where needed. A system was tested, therefore, which would provide bus service on eleven routes. Express buses would serve essentially every radial corridor considered in the auto-dominant system of highways.

Critical Routes.—The most critical portions of the all-bus system would be the two routes serving the northwest and northerly corridors, respectively.

No freeway is yet planned in either critical corridor within the District lines. It was found, however, that even with a complete transit system, freeways would be required in the vicinity of Wisconsin Avenue penetrating the District as far as the Intermediate Loop and from the Silver Spring-Wheaton area through the entire north portion of the District to a connection with the Inner Loop Freeway. An eight-lane north central freeway would be so heavily used by general traffic by about 1970 that the buses required on such a route could not be operated in the traffic stream without jeopardizing the speed and dependability of the transit service.

Special Bus Roadways.—The all-bus system, therefore, contemplates two special highway lanes for the exclusive use of buses in certain sections of each of these routes. Preferably, these would be built in a fully grade separated roadway along a valley or other natural separator between neighborhoods. Such a proposal could conceivably raise insurmountable objections from the standpoint of esthetics and other considerations. One alternative would be to operate the buses indefinitely on surface streets with such privileges as could be accorded them to speed their operation. Another alternative would be to build several miles of bus subways. The cost of such subways would be approximately $20,000,000 per mile, however, which is hardly realistic considering their relatively low passenger-carrying capacity if any station stops are made en route.

Consideration was given to bus operation on private roadways in a wide center mall of the north central freeway as a long-term, or even permanent, solution to the rapid transit requirements of the area. There were compelling reasons, however, for recommending against such a scheme. The number of buses in rush hours after the first few years would be greater than could be operated with buses stopping at intermediate stations. With private bus roadways in a center mall, a rate of flow of more than about 120 buses per hr in one direction would require two extra bus lanes for non-stop buses or else multi-platform stations that would necessitate awkward and probably infeasible bulges in the center mall, comparable with barrier-type turnpike toll plazas.

With buses operating in mixed traffic on a freeway, this situation is met by having only one-half or one-third of the buses stop at any one station. The balance pass that station as non-stop or skip-stop buses. There would be capacity for buses to operate in this manner to at least 1980 on the freeways proposed for rapid transit bus service in the recommended scheme—Plan IV.

It must constantly be kept in mind that even the target date of 1980 is a relatively short-term objective in a region that may continue to grow in population for decades or even centuries. The future welfare of the region would be jeopardized by embracing a plan which threatened to become inadequate within a few years.

Effect of Surface Bus Operation in Sector Zero.—It is not possible to estimate with absolute accuracy the number of automobiles and trucks that will be moving on the streets in the central area during peak hours in 1980. The Inner Loop Freeway and other bypasses are expected to accomplish a marked reduction in through traffic on surface streets. The number of terminating and originating vehicles, on the other hand, will probably increase.

The civil engineering consultant considered it likely that by 1980 the number of buses required by the all-bus system would result in movement at walking speeds or less during rush hours. The service would be slowed even in off-peak periods by the many complex and closely spaced intersections, as well as by the interference of autos and pedestrians. This problem would be even greater in cities with more intensely developed central business districts.

This conclusion is verified by the experience in several cities where express buses are now operating on freeways in the manner proposed for Washington under the all-bus plan. High operating speeds on the freeway portions of the routes are tempered by low speeds in the congested downtown areas so that the average overall speed in most cases is not impressive, as shown in Table 1.

Under the recommended bus-rail plan, the average speeds on bus routes should be satisfactory. The proposed rail facilities would greatly reduce the number of buses otherwise required. Thus, the remaining buses in Sector Zero would be able to move at reasonable speeds.

Consideration was given to off-street bus terminals, to bus subways in the central area, and to distribution of bus passengers from the outer limits of the downtown area by moving belt or other type of mass transportation facility. None of these schemes proved practical, however, because of the large area

TABLE 1.–UTILIZATION OF FREEWAYS BY URBAN TRANSIT BUSES IN VARIOUS CITIES[a]

City	Bus Route	Length of Route			Number of Stops on Freeway	Rush Hour Intervals in Minutes[b]	Rush Hour Service Bus Speeds, in Miles Per Hour		
			Part of Route on Freeway				Express		Local
		Terminal to Terminal	Miles	Percentage			On Freeway	Terminal to Terminal	Terminal to Terminal
Atlanta, Ga.	6 Lines	11.5	-	-	0	3	-	17	10
Chicago, Ill.	#2--Hyde Park	8.5	5.5	65	0	7	36	23	16
	#5--Jeffery	14.0	5.5	39	0	3	36	13	10
	Outer Drive Express	11.5	6.5	37	0	3	38	13	10
	LaSalle Express	7.6	4.0	53	0	2	38	11	9.2
Cleveland, Ohio	#39--Memorial Shoreway	-	8.5	-	0	3	21-24	18-20	-
	#55--Bulkley Blvd.	-	5.5	-	0	1-1/2	13	13	-
Dallas, Tex.	5 Lines	-	1.3-3.7	-	0	5-10	-	-	-
Detroit, Mich.	#33--Dearborn	-	3.8	-	0	N.A.	32	18-20	14-15
	#41	-	6.7	-	0	N.A.	40	18-20	14-15
	#58--Plymouth	-	4.6	-	0	N.A.	19.2	18-20	-
Los Angeles, Calif.	#44--Beverly	11.0	3.1	28	0	N.A.	27	12	10
	#83	10.0	3.1	31	0	N.A.	27	11-14	-
	#91	15	5.2	35	0	N.A.	27	15	12.5
	#93--Express	-	8.0	-	3	N.A.	23-30	18-20	-
	#93--Local	-	4.4	-	2	N.A.	23-28	15-17	-
	#58	-	15.0	-	0	N.A.	28	-	-
	#5	15.6	6.6	42	3	N.A.	36	17-21	11.2
	#7	-	5.6	-	0	N.A.	37	-	-
Pittsburgh, Pa.	#22, #23 and #24	-	-	-	0	N.A.	-	16.5-25.4	11.6-15.9
Oakland, Calif.	Key System Transbay Lines (L)	13.69(L)	-	-	0	2-6	43[c]	20-26	-
San Francisco, Calif.	#30	-	3.6	-	0	N.A.	-	-	-
	Greyhound to:								
	San Rafael	13	-	-	0	8	-	28	-
	San Mateo	11	-	-	0	N.A.	-	-	-
	Redwood City	17	-	-	0	N.A.	-	-	-
	San Jose	49	21	-	0	N.A.	-	32	-

[a] Compiled by De Leuw, Cather & Company from data in report "The Utilization of Freeways by Urban Transit Buses: A Nation-Wide Survey" by Wolfgang S. Homburger and Norman Kennedy--The Institute of Transportation and Traffic Engineering--University of California, Berkeley. [b] N.A.-Not available. [c] On Key System's "L" Line as observed by De Leuw, Cather & Company on April 18, 1958.

of the business district. Many route-miles of expensive facilities would be required to cover the area adequately with bus subways or mechanical devices. A centrally-located off-street bus terminal would have to be about as large as the Port of New York Authority bus terminal in Midtown Manhattan (New York, N. Y.), and its approaches would require several thousand feet of fully grade-separated bus roadways, either subway or elevated.

Conclusions.—For the reasons set forth previously, it was urged that the use of express buses be limited to those routes included in the recommended system. In the stage development of this system, however, express buses should be used on surface streets and on freeways as the latter become available. This procedure would initiate fast service at an early date to the areas which would later be served by rail rapid transit. Thus, the system in effect in 1965 would be the all-bus plan but without any separate bus roadways. It would be reasonably adequate, in the opinion of the consultant, to serve the estimated patronage of that era.

PLAN III—ALL-RAIL RAPID TRANSIT PLAN WITH RECOMMENDED SYSTEM OF HIGHWAYS

Washington's geography as well as its low-density pattern of residential, commercial and government office building developments discourages widespread use of rail rapid transit. The high initial cost of such facilities, even if built in the center mall of a freeway, requires concentrations of travel not found in Washington except in a few instances. Characteristics of the region tending to prevent heavy flows of transit traffic along a single route include the large areas devoted to public or institutional use; the vast extent of parks, waterways, ravines and otherwise uninhabited areas; and, despite noteworthy apartment buildings, the extensive areas devoted to single-family dwellings on large lots. In the central area a similar lack of concentration prevails. The all-rail rapid transit plan proved to be grossly over-designed, therefore, to meet the needs of 1980. It was recommended, however, that thought be given in planning all radial freeways to the ultimate need for rail rapid transit, or a similar type requiring a private right-of-way. In some instances, actual need may not develop for many years. Unless space for such facilities is included in the initial stage of construction, the cost of providing for them later may be prohibitive.

ESTIMATED CAPITAL COSTS

Estimates were made of the capital cost of each of the four basic systems studied—auto-dominant; all-bus rapid transit coordinated with freeways; all-rail rapid transit coordinated with freeways; and recommended highway-transit system (Table 2). Recommendation of Plan IV over the other schemes, however, was based on practical considerations of speed, capacity, and other physical problems rather than on cost.

Estimates of revenues and of operating and maintenance expenses were made by the civil engineering consultant. Proposals for financing both highway and transit systems were prepared by other consultants.

TABLE 2.—CAPITAL COSTS AND OTHER DATA ON FOUR ALTERNATE TRANSPORTATION SYSTEMS TO SERVE ESTIMATED 1980 TRAFFIC VOLUMES IN THE NATIONAL CAPITAL REGION

	Estimated Capital Cost in Thousands of Dollars			
	Plan I Auto-Dominant	Plan II All-Bus	Plan III All-Rail	Plan IV Recommended
Highway Facilities:				
Freeways	$1,870,450	$1,401,400	$1,401,400	$1,401,400
Parkways	299,050	220,150	220,150	220,150
Major Streets	201,650	181,450	181,450	181,450
	$2,371,150	$1,803,000	$1,803,000	$1,803,000
Parking Facilities:				
Outer Transit Stations	$ -	$ 20,800	$ 20,800	$ 20,800
Intermediate Stations	-	16,800	16,800	16,800
Sector Zero (Downtown)	236,000	119,000	119,000	119,000
	$ 236,000	$ 156,600	$ 156,000	$ 156,600
Total Highway and Parking Facilities	$2,607,150	$1,959,600	$1,959,600	$1,959,600
Express Bus Facilities:				
Special Bus Roadways	-	$ 57,100	-	$ -
Outer Terminals and Freeway Stations[a]	-	31,050	-	30,600
Shops and Garages	-	13,500	-	8,650[c]
Rolling Stock (Buses)	-	45,000	-	28,750[c]
		$ 146,650		$ 68,000
Rail Rapid Transit:				
Subways	-	-	$ 561,700	$ 313,150
Open Construction	-	-	176,550	85,050
Outer Terminals	-	-	8,500	3,950
Yards and Shops	-	-	20,000	12,250
Rolling Stock (3-Car Units)	-	-	72,000	44,100
			$ 838,750	$ 458,500
Total Transit Facilities	-	$ 146,650	$ 838,750	$ 526,500
Total System Cost	$2,607,150	$2,106,250	$2,798,350	$2,486,100

	Physical Features			
	Plan I Auto-Dominant	Plan II All-Bus	Plan III All-Rail	Plan IV Recommended
Highway Facilities:				
Route-Miles of Freeways and Parkways	344	326	326	326
Factor	100	95	95	95
Lane-Miles of Freeways and Parkways	2,032	1,775	1,775	1,775
Factor	100	87	87	87
Net Additional Parking Spaces Needed:				
Outer Transit Stations	-	20,800	20,800	20,800
Intermediate Stations	-	11,200	11,200	11,200
Sector Zero (Downtown)	85,000	48,500	48,500	48,500
Total	85,000	80,500	80,500	80,500
Factor	100	95	95	95
Transit Facilities:				
Express Bus				
Number of Routes	-	11	-	8
Route-Miles[b]	-	103.8	-	66.4
Buses Required	-	1,800	-	980
Rail Rapid Transit:				
Number of Routes	-	-	9	4
Route-Miles[b]				
Subway	-	-	32.4	14.3
Open-to-the-Sky	-	-	44.8	20.1
Tree-Car Units Required	-	-	400	245

[a] Excluding parking facilities. [b] Includes duplication where two or more routes operate on same street or right-of-way. [c] More buses would be required in 1965 than in 1980, since rail rapid transit would replace some of the express bus routes during this period. As buses purchased prior to 1965 became worn out, therefore, they would not all be replaced. The depreciation funds represented by such excess buses would presumably be invested in rail rapid transit cars. Garages and bus shops, which would have a longer life than the buses, should be so designed that they could be partially converted for use in servicing the rail cars or used by the expanding fleet of local buses.

PRESERVING CAPACITY FOR THE FUTURE

The pattern of future growth of the National Capital Region will be influenced strongly by the nature and quality of its transportation system. The cornerstone of the population and employment distribution studies, however, was the assumption that the entire region will ultimately be covered with an adequate transportation network. The area can then develop under the influence of such things as topography and access to public utility services without considering the variable of transportation. This was a reasonable approach to that phase of the problem, since it is always desirable to hold one factor constant while measuring the influence of variations in others.

The freeways built early in the program, however, will have a profound influence on the pattern of growth of the metropolitan region. They will encourage fast and intense settlement in the areas directly served, delaying the growth of areas lacking such freeways. Evolution of the residential and industrial development in this manner would simply repeat what has happened in every large city, including Washington, as new transportation arteries were built.

Facing this very realistic set of circumstances, it seems wise to strive for flexibility and reserve capacity in development of future transportation arteries. Rail rapid transit to certain sections of the area was shown to be justified as the ultimate plan by estimated patronage. These findings were based largely on existing and anticipated development. Meanwhile, if highway facilities are built to other sectors of the metropolitan area, but not to those for which rail rapid transit was recommended, need for recommended lines may be delayed, and need for rail facilities to other areas may develop. Final decisions on construction of rail rapid transit, therefore, should be made in conjunction with adoption of a complete freeway program.

Even after a complete freeway program is set, administrative decisions will determine the sequence in which various highways are built. This construction schedule will have a strong modifying effect on the distribution of population and perhaps also on the location of job opportunities. The first freeways built, therefore, may be overcrowded long before 1980, and express bus service may have to give way to rail rapid transit to meet transportation demands. While the proposed plan is intended to serve a population of 3,000,000, the freeways will still be in use even if the population grows to 4,000,000 or 5,000,000. Rights-of-way for additional freeways will be difficult to find.

It was recommended, therefore, that in the planning of future radial freeways a cross section similar to that shown in Fig. 3 be provided to afford maximum flexibility and to provide reserve capacity for automobile and truck traffic as well as for the mass movement of people. Under this plan there would be a three lane or four lane roadway for traffic in each direction. These roadways would be separated by a 64-ft mall with 51 ft from center-to-center of the piers supporting cross-street bridges. In the first stage, this wide mall would be landscaped and held available for future developments. Public transportation in this stage would consist of express buses operating in the general traffic lanes. They would make stops at appropriate intervals on the parallel service roads without special station facilities or at simple stations within the end spans of the cross-street bridges.

As automobile and truck traffic became heavier, the center mall could be developed with a three-lane vehicular roadway to be used by one-way traffic. Direction of flow would be reversed between the morning and afternoon rush periods. This would be consistent with policies of the Bureau of Public Roads,

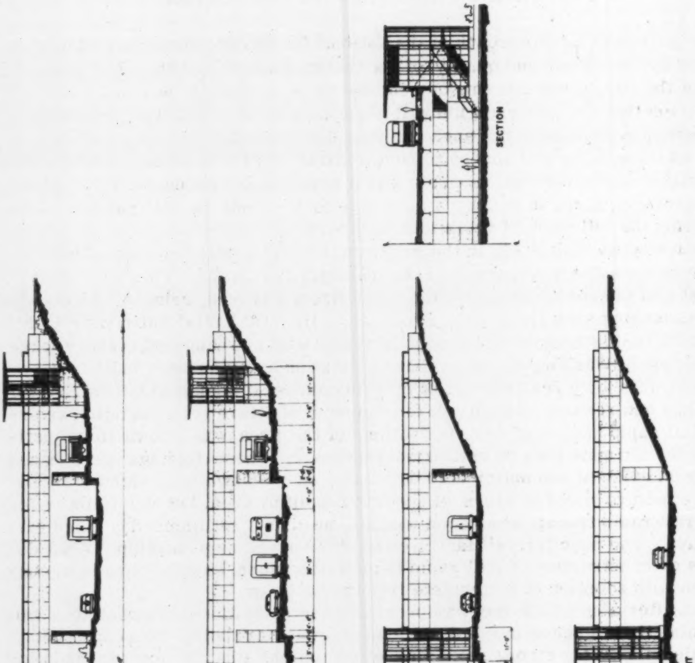
SECTION
1ST STAGE
2 ND STAGE
3RD STAGE
ULTIMATE DEVELOPMENT

FIG. 3.—TYPICAL CROSS SECTION THROUGH FREEWAY SHOWING STAGES OF DEVELOPMENT

United States Department of Commerce. Presumably, express bus traffic would be much heavier in this stage, and buses might almost preempt one lane in each direction on the initial roadways of the freeway.

In the third stage, a portion of the express bus traffic would be removed from the freeway and operated on a private roadway in the center mall. The bus roadway would be reduced to two lanes at the stations in order to provide adequate platforms for passenger loading and unloading. Stairways or escalators would connect with stations on the cross-street bridges where passengers from local feeder buses would transfer.

In the fourth stage of development, the bus roadways could be displaced by a double track rail rapid transit facility which would provide capacity for any volume of transit patronage likely to develop within the next century.

This plan of stage development on radial freeways still in the planning stage would permit adequate facilities to be provided to meet any likely eventuality without requiring prohibitive outlays of capital in the early years. This plan is particularly adaptable to the proposed interstate freeway to the northeast, which is now in the preliminary stages of planning; to the route extending westerly into Virginia on an alignment lying one to two miles north of U. S. 50; and to the John Foster Dulles Airport route. Other routes for which this plan should be considered include Shirley Highway, when this freeway is rebuilt and widened, and the proposed new freeway to the southeast.

CONCLUSIONS

Many years will be required to build all of the freeways and parkways needed not only in the National Capital Region, but in all other large cities. Meanwhile, people are proceeding at a frantic pace to build homes, shopping centers, and factories on any land that is available. Unless rights-of-way are reserved promptly for the needed highways, it may become prohibitively expensive to build many vital sections of each city's network. Great emphasis should be placed, therefore, on the adoption of an official plan for highways not only for the immediate future, but for a generation hence. Rights-of-way should be preserved which will be sufficiently wide for any transportation purpose which is at all likely to develop in the decades to come.

DISCUSSION

KENETH M. HOOVER.[2]—Mr. McConochie's conclusions seem to be slanted toward the acquisition of rights of way for future highways alone. The writer thinks that acquisition of rights of way for transit were just as important and that this was the intent of the majority who worked on the program.

More important to the writer is the fact that here, for the first time, an objective study was made to ascertain the total transportation requirements of a metropolitan community and that the study resulted in a plan that brought about action to organize the community to cope with the problem.

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[2] Chf. Engr., San Francisco Bay Area Rapid Transit District, San Francisco, Calif.

Since Mr. McConochie wrote this paper, Congress has taken action, and the President of the United States has signed a bill providing for the creation of a Federal agency to modify and carry out the basic findings of the study. The writer has no doubt that this agency will enlarge and refine the findings of the planners, economists, engineers, and others who conducted the present study.

S. D. FORSYTHE.[3]—The author very ably condenses the great mass of data gathered in connection with the Transportation Plan for the National Capitol Region, so that the important considerations offered in support of each of the four plans are brought into reasonably sharp focus. The conclusions reached by the planning groups represent a middle ground between the two extreme plans, the Auto-Dominant Plan and the All Rail Rapid Transit Plan with Recommended Systems of Highways, and as such (that is, a middle ground), it must be conceded that their position is sound. However, the thoughts expressed that "Express buses would be used in the early years on the routes ultimately to be served by rail rapid transit," and that "Rail Rapid Transit to certain sections of the area was shown to be justified as the ultimate plan by estimated patronage" is disturbing and ambiguous. It suggests that either the authors of the plan look upon rail rapid transit (and by this term it is assumed they include all types of multiple car operation) as a last resort that may never have to be installed at all on these routes, or else, if they do expect rail rapid transit to be installed by 1980, on these routes, they have not considered the capital investment in the so called second and third stages (as illustrated in Fig. 3) that must be written off in a short span of years. Neither have they considered the staging problems attendant upon maintaining an express bus line operating at or near capacity while converting the median strip or private right-of-way from bus roadways and stations to rail rapid transit facilities.

If the planners are satisfied that some form of rail rapid transit will be justified on some routes by 1980, then it seems wiser to plan immediately to install those facilities at the earliest possible time than to plan three different periods of intense and expensive construction before the ultimate plan is realized.

The ability of modern rapid transit to attract riders is well illustrated by the Congress Street (expressway) route in Chicago, Ill. Today, this route, serving the same territory the old Garfield Park elevated line served with competing rapid transit lines on either side of it, now as then, is carrying 50% to 65% more passengers in the rush hour periods than the old route.

Furthermore, this line operating between 30% and 40% of capacity is carrying 150 people in the peak periods for every 100 people the adjoining eight lane expressway, operating at between 80% and 90% capacity, is carrying in the same periods. On the basis of investment per passenger capacity, rail rapid transit facilities should prove much less costly than the three stage plan proposed.

One last point seems worthy of further thought. Various types of multi-car rapid transit systems are being given more intense study in the United States and in Europe at the present time than at any previous time in this century. Washington D. C., is in an excellent position to profit by, and indeed to contribute greatly, to these studies. Therefore, it seems quite premature to assume that articulated rail cars will prove to be the most satisfactory rapid transit vehicle to use in Washington, where there is no compulsion to conform to any existing types. As the nation's capitol, it should be equipped with the finest type of system the nation can produce, and it is to be earnestly hoped that the same combined effort on the part of so many able people will characterize the actual building and operating of the adopted plan.

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[3] General Supt. of Engrg., Chicago Transit Authority, Chicago, Ill.

NATHAN CHERNIACK,[4] F. ASCE.—Mr. McConochie presents a case study on urban transportation for a given metropolitan area, Washington, D. C. and its environs. Its value is enhanced considerably by the fact that it consists of several distinct studies of alternate methods that might solve the transportation problems of metropolitan Washington. Each separate solution involves its own economic cost with correlative physical advantages and disadvantages.

There have been similar urban transportation studies for specific cities to meet their respective needs. Few, however, have been reported in technical journals with the view of distilling out general urban transportation principles. Out of the urban transportation study for metropolitan Washington, McConochie has made two contributions. He has sharpened up the technical language, something sorely needed before the controversial issues involved in the general topic of urban transportation can be resolved. He has also indicated some general principles that the engineering profession would do well to adopt in appraising urban transportation needs in other metropolitan areas.

For example, the author states that "Throughout this paper, the terms 'express' and 'rapid transit' are used interchangeably to indicate services by buses on freeways and parkways, as well as rail vehicles on private rights of way." From current literature on urban transportation, one gets the impression that there are only two choices possible for meeting urban transportation needs. One is to adopt transportation on the highways in autos, with very limited capacities to handle people in rush hours; the other is to make use of rapid transit on rails on private rights of way, with tremendously high capacities to handle people in rush hours, even if population densities in tributary areas along the rail alignments are such as to preclude the possibility of ever utilizing a fraction of the designed capacities. The possibility that there is a third choice, that much more widespread use could be made of express buses on freeways and parkways having sufficiently high passenger carrying capacity to rapidly transport masses of people in rush hours, appears to be completely absent from current literature on urban transportation.[5] McConochie's treatment of the terms "express" and "rapid transit" as interchangeable and as describing passenger transport services either by rail vehicles, or private rights of way, or by buses on freeways and parkways is indeed refreshing and reflective of advanced thinking on modern urban transportation.

After analyzing the Auto-Dominant plan for which autos would meet "the major portion of the travel needs of the metropolitan area," and "public transportation in the form of local buses operating on the surfaces streets giving service of the type and relative extent provided in 1955," as "transit playing a minor role," McConochie comes to these conclusions with respect to the Washington metropolitan area. "There are serious engineering questions as to the practicability of the autodominant plan." "A transportation policy requiring the National Capital Region to be served almost exclusively by automobiles would be incompatible with the distribution of living, shopping, and working places of the area's inhabitants as foreseen by the staff of the Commission and Council."

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[4] Economist, Port of N. Y. Authority, New York, N. Y.

[5] "Passenger Data for Urban Transportation Planning," by Nathan Cheriak, Proceedings, ASCE, Vol. 85, No. HW4, December, 1959.

In effect, the author is saying that to set up an auto-dominant plan for any urban area is to set up a "straw man." Few large urban areas could get along without a substantial proportion of their rush hour passenger movements being handled by some type of mass transit.

On the other hand, the author points out "that a good rapid transit system (as he defines it) can complement a network of urban freeways, but it cannot provide an adequate substitute for modern highway facilities." "Even an over-designed system of rapid transit facilities would not drastically reduce the need for freeways." "This highway system (an enlarged highway network) would be necessary in connection with either the all-bus or the all-rail transit plan." "The rapid transit system involving the lowest investment for special transit facilities would be comprised of express bus routes on freeways built for general traffic, with separate lanes for buses where needed."

The recommended Plan IV recognizes the need for mass transit but does not recommend all-rail transit exclusively. It recommends 66 route miles of express buses on freeways and parkways and 34 miles of rail rapid transit to meet metropolitan Washington's transit needs of the future. Because an enlarged freeway and parkway system is essential even with a transit plan, why not utilize the freeways and parkways exclusively, as a modern transit system with express buses? Why recommend the sinking of additional new capital in complementary over-designed rail transit system when surely, for most of the proposed rail transit mileage, the anticipated rush hour passenger density in the next two decades would not demand a fraction of the available rail transit capacity that would be provided? At the same time, express buses on freeways would meet all the transit needs for the next two decades.

The author, in fact, states that "Consideration was given to bus operation on private roadways in a wide central mall of the north central freeway as a long term or even permanent solution to the rapid transit requirements of the area." He goes on to say that "There would be capacity for buses to operate in this manner (having only one-half or one-third of the buses stop at any one station; the balance pass that station as non-stop or skip-stop buses) to at least 1980, on the freeways proposed for rapid transit bus service in the recommended scheme - Plan IV." But he says further, "it must be constantly kept in mind that even the target date of 1980 is a relatively short time objective in a region that may continue to grow in population for decades or even centuries." True!

Instead of recommending the sinking of new capital in rail transit now, as incorporated in Plan IV, why was the sounder recommendation not incorporated in Plan IV? There, McConochie states, that "it is recommended that in planning future radial freeways, a cross section similar to that shown in Fig. 3 be provided to afford maximum flexibility and to provide reserve capacity for automobile and truck traffic, as well as for the mass movement of people." "In the fourth stage of development, the bus roadways could be displaced by a double track rail rapid transit facility which would provide capacity for any volume of transit patronage likely to develop within the next century."

Hence, the final query: Why recommend the building of a rail transit system now, in the first stage, when it is possible to so plan freeways and parkways for stage construction that rail transit could be provided when a real need has been conclusively demonstrated through usage of the freeways and parkways in the several stages, by express buses, preceding the final conversion to rail transit?

WILLIAM R. McCONONCHIE,[6] F. ASCE.—Mr. Cherniack has made a very convincing presentation of the case for express buses on freeways and parkways as a permanent plan for providing rapid transit in metropolitan areas. This type of operation was carefully considered and recommended for several of the lighter routes in the Washington, D. C. study. The writer believes, however, that the findings relative to use of buses on the heavier routes were not only valid for Washington, but also for other cities of comparable size. For the four heaviest corridors in Washington, rail rapid transit was recommended rather than express buses for the following reasons:

1. The freeways on which center mall rail rapid transit was recommended would be so heavily used by automobiles and trucks soon after completion that introduction of 200 buses per hour to 250 buses per hour in each direction during rush hours would make intolerable a situation that will be difficult at best.
2. Maintenance of satisfactory speed and regularity of bus service on a badly overloaded freeway is virtually impossible. People then turn to the use of their own automobiles to gain the flexibility needed to choose less congested routes, and bus patronage quickly deteriorates to the point at which no bus service is needed.
3. Express buses arriving at the downtown end of the route must either be absorbed in the general traffic stream on the surface streets, that is not an inviting prospect under today's worsening traffic conditions, or they must have a terminal such as that operated in New York City by the Port of New York Authority with scores of bus berths as well as a subway system to distribute the passengers. In Washington, it was estimated that local buses, plus the express buses from the routes recommended for this type of operation would use all of the available space on the surface streets by 1980. It was concluded, therefore, that compounding downtown congestion with additional buses would be unwise.
4. Once a freeway is opened to traffic it is incumbent on responsible officials to keep it in operation under any and all circumstances. This policy must be even more rigidly adhered to in connection with a public transportation facility because its patrons have fewer alternatives and the operating company has little flexibility. There might be insurmountable problems, therefore, in temporarily transferring an express bus line from the center mall of an expressway to roadways carrying general traffic while the center mall was being converted for rail rapid transit operation.

The discussion accorded this paper is sincerely appreciated. In the opinion of the author, the points raised by Hoover, Forsythe, and Cherniack are not so much indicative of differences of opinion on what is obviously a very lively topic, but rather they are vivid illustrations of the careful and exhaustive studies that must follow the preliminary planning of a comprehensive transportation system. As the project is carried to advanced stages of planning, each step will have to be considered in the light of recent developments and experience in this fast-moving field.

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[6] Dir., De Leuw, Cather and Co., Chicago, Ill.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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SHOCK PRESSURES EXERTED BY BREAKING WAVES ON BREAKWATERS

By Shoshichiro Nagai,[1] M. ASCE

SYNOPSIS

The results of the studies from April, 1956 to March, 1959 of the shock pressures exerted by breaking waves on composite-type breakwaters are presented. Such hitherto unknown problems as relationships between wave profiles and pressures from instant to instant in plunging of breaking waves into the breakwaters, the velocities of water particles in the breakers, and the behavior of air bubbles entrained in them, were clarified by the synchronised use of a 16 mm high speed movie taken at 3,000 frames per sec and an electro-magnetic oscillograph. A formula for the prediction of the maximum shock pressures and the maximum resultant pressure per unit length of breakwaters were derived.

INTRODUCTION

Since the prediction of shock-pressures exerted by breaking waves on breakwaters is the most important thing to be known for the design of breakwaters, the problem has been studied by a number of harbor engineers and in-

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vestigators during several decades in nature as well as in models in many parts of the world. But very little information has been available in regard to both the theory and the practical data of the shock pressures obtained by measurements in seas, because of the difficulties in the experiments and actual measurements due to the high intensity and instantaneousness of the shock pressures. To the author's knowledge, it seems that there exists no generally accepted formula which is available with sufficient confidence to insure that both safe and economical designs are obtained, being established on the basis of a sufficient number of practical or experimental data.

The author has been studying this subject since 1956 and the experiments are still under way. Over two thousand tests of the shock pressures exerted by breaking waves on the vertical walls of composite-type breakwaters by the use of two or three wave-pressure-gauges have been performed, and the behavior from instant to instant of the breaking waves plunging against the breakwaters, the relationships between the wave pressures and the configuration of the breaking waves, the motion of the water particles in the breaking waves, etc., were measured by means of a 16 mm high speed movie taken at 3,000 frames per sec in synchronism with the pressure gauges. On the other hand, the mechanics of the shock pressures was also studied theoretically. From the results of the theoretical considerations and the experimental data, the new formulas of the maximum shock pressure as well as the vertical distribution of maximum simultaneous shock pressures were established for the vertical walls of composite-type breakwaters.

PROMINENT PAST RESEARCH

Hiroi's Formula.[2]—Owing to I. Hiroi's formula the intensity of the pressures exerted by breaking waves on breakwaters are expressed by the equation, $p = 1.5\, w_o\, H$, in which w_o represents the unit weight of water and H is the wave height. The pressure expressed by Hiroi's formula corresponds to a dynamic pressure exerted on a vertical wall by the fall velocity and orbital crest velocity of a water particle when the water particle falls down on the still water surface at an angle of 45° from the breaking wave crest. This formula has been used for about forty years in Japan for the calculation of the resultant of shock pressures acted on the unit length of a vertical wall by employing the conventional equation, $P = p \cdot h$, in which h is the height from the top to the bottom of the vertical wall of a breakwater. As is seen from this equation, it is assumed that the shock pressures are uniformly distributed with an average value of p over the total height of the vertical wall. This assumption was obtained from actual experience in a number of composite-type breakwaters in Japan where the tops of the vertical walls of breakwaters have generally been constructed as high as 2 m to 2.5 m above the high water level of the ordinary spring tide. Owing to the experimental results, it was proved that the values of P determined by the conventional method were acceptable in a rough approximation for such low composite-type breakwaters as have been constructed in Japan.

Larras' Experiments.[3]—J. Larras probably gave the first information concerning the time variation of shock pressures exerted by breaking waves on vertical walls built on slopes. The pressure-time curves obtained in his experimental wave tank were the same types as those shown subsequently in Figs. 10 (a) and (b).

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[2] "The Force and Power of Waves," by Isamu Hiroi, Engineering, August, 1920.

[3] "Le Déferlement des Lames sur les Jetée Verticales," by J. Larras, Ponts et Chaussees, May, 1937.

Bagnold's Study.[4]—R. A. Bagnold measured the shock pressures of breaking waves exerted on a vertical wall by a single wave made to break by means of a sloping beach. But he did not investigate the relationships between the shock pressures and the characteristics of waves, as well as the vertical distribution of pressure. After assuming that the shock pressure is caused by an adiabatic compression of the cushion of air enclosed between a vertical wall and the front face of a breaking wave, he established an equation which describes the relationships between the shock pressure and time. It was proved, theoretically as well as empirically, that his equation did not truly describe the phenomenon of shock pressure.[5]

Minikin's Equation.[6]—Using Bagnold's assumption, R. R. Minikin derived the following equation for maximum shock pressures in tons per sq m caused by breaking waves on the vertical breakwaters:

$$p_{max.} = 102.4\ h_1 \left(1 + \frac{h_1}{h_o}\right) \frac{H}{L} \quad \ldots\ldots\ldots\ldots\ (1)$$

He assumed the maximum pressure $p_{max.}$ to occur always at the still water surface and the vertical distribution of shock pressures exerted on the vertical wall in the same occurrence as $p_{max.}$ to be expressed by

$$p_y = p_{max.} \left(1 - \frac{2y}{H}\right)^2 \quad \ldots\ldots\ldots\ldots\ (2)$$

in which p_y is a pressure intensity at a distance y above or below the still water surface.

Eq. 1 is considered to be inadequate in its derivation,[5] and Eqs. 1 and 2 seem to have been derived mostly from Minikins experience with damage in actual breakwaters. As will be shown later, the author's experimental results proved that the values calculated by Eq. 1 from the experimental values were within ± 60% for the ordinary shock-pressures of breaking waves, but Eq. 1 indicated very great discrepancies for the extraordinary high shock-pressures. The pressure-distribution expressed by Eq. 2 was proved the same as the A-type distribution defined by the author.

The resultant of the total pressures exerted on the unit length of a vertical wall, P, is calculated by the equation

$$P = \frac{1}{3}\, p_{max.}\ H + w_o\, \frac{H}{2} \left(h_1 + \frac{H}{4}\right) \quad \ldots\ldots\ldots\ (3)$$

in which the first term in the right hand side of Eq. 2 expresses the dynamic wave-pressure and the second the hydrostatic pressure corresponding to the depth of water at the crest position of the wave.

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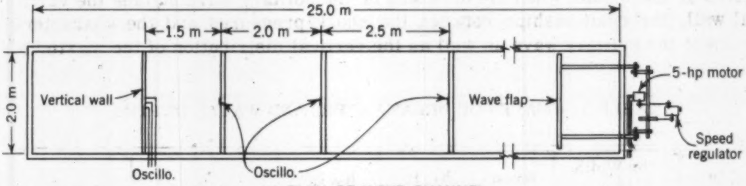
[4] "Interim Report on Wave-Pressure Research," by R. A. Bagnold, Journal, I. C. E., Vol. 2, 1938-1939.

[5] "Researches of the Shock Pressures of Breaking Waves on Breakwaters," by S. Nagai, Interim Report, Proceedings, 5th Conference of Coastal Engrg., Japan, November, 1958.

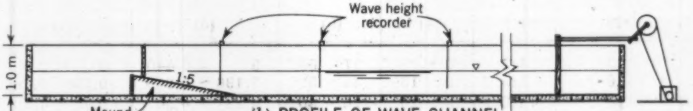
[6] "Winds, Waves and Maritime Structures," by R. R. Minikin, London, 1950.

EXPERIMENTAL EQUIPMENT AND PROCEDURE

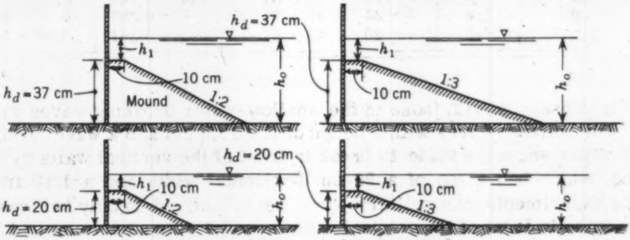
The experiments were carried out in a concrete channel with an overall length of 25m, a width of 2m, and a depth of 1m, at the Field Hydraulic Laboratory of the Osaka City University, as shown in Fig. 1. In order to permit visual observation as well as photographic measurement, the one side of the wave channel in the neighborhood of breakwater-models was made of 10-mm plate-glass. The experimental waves were generated by a flap-type wave-generating machine, and the periods of the waves were changed T ≑ 1.2 to 1.3 sec., 1.5 to 1.6 sec., and 1.9 to 2.0 sec., the heights H ≑ 6 to 22 cm., and the lengths L ≑ 2.0 to 4.5 m. These waves were made to break in front of the breakwaters and to strike severely against the vertical walls by the slopes of rubble-mound which were constructed with wood in the inclination to the horizontal bottom of 1:2, 1:3, and 1:5 as the foundation of the vertical walls. The period, length, and height of the waves were measured by three electric wave-height gauges located at spaces of two and three meters as shown in Fig. 1. The measurement of the shock-pressures of breaking waves were performed by the use of the two or three strain-gauge type pressure-recorders devised specially for this experiment in the laboratory in 1953. The pressure-recorders, which have a phosphor-bronze thin circular plate of a 4-cm diameter, were fed by 5,000 cycles A. C. so as to measure shock-pressures changes occurring within $\frac{1}{500}$ sec.


25.0 m
1.5 m
2.0 m
2.5 m
2.0 m
Vertical wall
Wave flap
5-hp motor
Speed regulator
Oscillo.
Oscillo.

(a) PLAN OF WAVE CHANNEL


Wave height recorder
1.0 m
1:5
Mound

(b) PROFILE OF WAVE CHANNEL


h_d = 37 cm
h_1
10 cm
Mound
1:2
h_d = 37 cm
h_o
h_1
10 cm
1:3
h_o
h_d = 20 cm
h_1
10 cm
1:2
h_d = 20 cm
h_o
h_1
10 cm
1:3
h_o

(c) TYPES OF BREAKWATERS

FIG. 1.—LABORATORY EQUIPMENT

When a 10-m horizontal width was constructed on the top of the rubble-mound (that is, in front of the vertical wall) with front slopes of 1:2 and 1:3, waves broke easier than in the case when the vertical wall was located on slopes of the same inclination with no horizontal top-width.

But when the slope of the rubble mound was 1:5, most waves broke as a plunging breaker and provided greater impact against the vertical wall than in the case of the 1:2 and 1:3 slopes. The slopes of breakwater-models and the water depths used in the experiments are summerised in Table 1.

Experiments of Solitary Wave.—As the breaker of a solitary wave has been often used for the study of the shock-pressures of breaking waves, some experiments concerned with a breaking solitary wave were performed in the wave channel as shown in Fig. 1. In order to compare the behavior of breaking waves at the instant when the breakers of the solitary wave strikes the vertical wall, the relationships between the shock-pressures and the characteristics of the solitary wave as well as the vertical distribution of the maximum simultaneous pressures with those in the shallow-water breaking waves were examined. The solitary waves with a height of H = 15.5 cm and a wave cerelity of $\omega = 2.30$ m per sec were made to break in front of the vertical walls by the base-mound, which had a top of a 20-cm horizontal width and a 1:10 front slope. The experiments of a solitary wave were confined to only two cases where the depths in front of the vertical wall, $h_1 = 4.4$ cm and 6.4 cm, and the depths above the horizontal bottom of the channel, $h_o = 41.4$ cm and 43.4 cm.

TABLE 1.—SHAPES OF BREAKWATERS AND WATER DEPTHS

h_d, in cm (1)	Top Width of Mound, in cm (2)	Slope 1:m (3)	h_1, in cm (4)	h_o, in cm (5)	h_1/h_o (6)	h_1/H (7)
37	10	1:2	0 ~ 29	37 66	0 ~ 0.440	0 ~ 2.0
37	10	1:3	9 ~ 15	46 ~ 52	0.196 ~ 0.289	0.55 ~ 0.78
20	10	1:2	6 ~ 20	26 ~ 40	0.231 ~ 0.50	0.75 ~ 1.70
20	10	1:3	9 ~ 15	29 ~ 35	0.310 ~ 0.428	0.55 ~ 1.63
37	0	1:2	0 ~ 24	37 ~ 61	0 ~ 0.393	0 ~ 2.0
33 ~ 38	0	1:5	3 ~ 20	40 ~ 57	0.075 ~ 0.351	0.30 ~ 1.0

BEHAVIOR OF BREAKING WAVES PLUNGING INTO BREAKWATERS

Configuration of the Water Surface of Breaking Waves before and after Striking against a Vertical Wall.

The Case of Shallow-Water Waves.—The behavior and configuration of the water surface before and after breaking waves plunged into the breakwater-models as shown in Fig. 1 (c) were studied by the use of a 16 mm high speed movie taken at 3,000 frames per sec for the waves of the characteristics and the water depths shown as follows:

a. $h_1 = 11.7$ cm, $h_1 + h_d = h_o = 48.7$ cm, T = 1.5 sec, H = 15 cm, L = 264, 264, 272 cm, peak pressure recorded on the pressure-gauge located at 5.7 cm below SWL = p_{peak} = 43, 59, and 75 gr per sq cm, peak pressure recorded at 7.7 cm below SWL = p_{peak} = 66, 45, and 61 gr per sq cm.

b. h_1 = 13.7 cm, h_o = 50.7 cm, T = 1.25, 1.38, and 1.67 sec, H = 18 to 19 cm, L = 220, 234, and 317 cm, peak pressure recorded at 10 cm below SWL = p_{peak} = 62 to 67 gr per sq cm, the maximum pressure = $p_{max.}$ = 131 to 141 gr per sq cm.

These shallow-water waves struck against the vertical walls of breakwater-models as plunging breakers, accompanied by spray 1m to 1.5 m high.

In the experiments of shallow-water waves, the measurement of pressure and wave-characteristics, as well as the photography were always performed

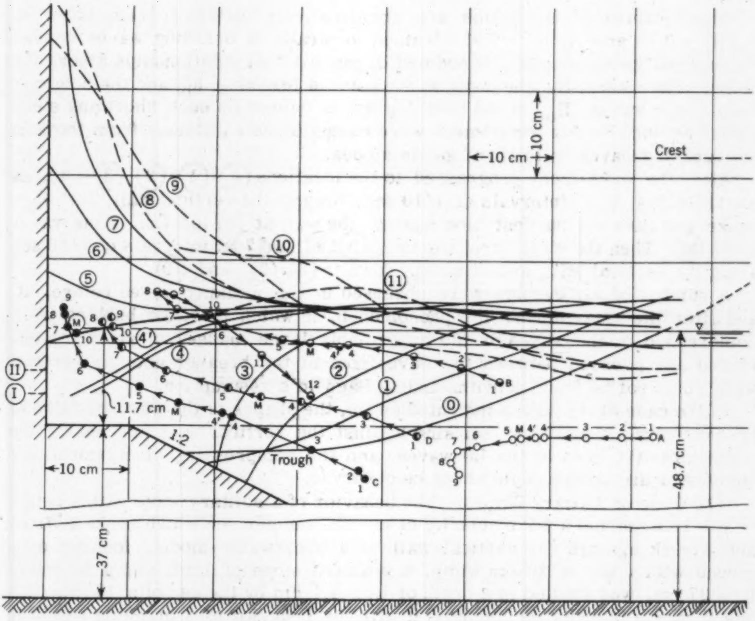


FIG. 2.—WATER SURFACE SHAPES OF A BREAKING WAVE

for several fully developed waves not effected by wave reflections from the vertical wall. The fifth wave in a series of generated waves appears to satisfy these conditions.

Fig. 2 shows the water surfaces or wave-profiles of the fifth wave at intervals of 1/10 sec obtained from the frames of the high speed movie taken for the waves indicated as (a). The paths of motion of water particles are also shown and T = 1.5 sec, L = 272 cm, and H = 15 cm. As is seen in Fig. 2, after the fourth wave which has been retrogressing from the vertical wall meets the coming fifth wave on the slope of the base-mound, it makes the wave-front of the fifth wave a bow-shaped curve with its lower part descending quickly down on the slope and its upper part pushing up the top of the fifth wave. At the instant when the fifth wave-crest reached the highest point as possible as it can rise and the wave-trough reached the lowest point, the wave-front stops to progress, indicating a height from SWL to the wave-crest, $H_u = 20$ cm and a height from SWL to the wave-trough, $H_l = 14$ cm. Putting $H_u + H_l = H_b$ (height of the breaker), $H_b = 34$ cm, therefore the ratios are obtained

$$\frac{H_u}{H_b} \doteqdot 0.59$$

and

$$H_l/H_b \doteqdot 0.41$$

These values of the ratios are comparatively different from those of $H_u/H_b = 0.75$ and $H_l/H_b = 0.25$ obtained generally in breaking waves caused on long and gentle slopes. It seemed in general that relationships among the height of breakers, H_b, the depth at the point of breaking, h_b, and the height of deep-water waves, H_o, in the breaking waves caused on such short and steep slopes as used in this experiment were comparatively different from those in the breaking waves on long and gentle slopes.

After the wave-front progressed to the locations ⓪-①-②-③-④, as shown in Fig. 2, at intervals of 1/10 sec, toward the vertical wall, the wave broke and gave the hardest blow against the wall at the location of the wave-front Ⓜ. Then the wave-front ran up to ⑤, ⑥, and ⑦ at intervals of 1/10 sec along the vertical wall, and then went down to ⑧, ⑨, and ⑩.

A number of air bubbles were observed in the breaking waves before, at, and after the breaking, but the phenomenon, in which those air bubbles were compressed in the waves or the appreciably large masses of air were enclosed and pressed between the wave-front of the breaker and the vertical wall, could not be found notwithstanding Bagnold's assumption.

In the case of the waves indicated as (b), the shapes of the wave-profiles of breakers before and after striking against the vertical wall are nearly the same as in the case of the (a) waves, and the compressing of entrained air bubbles or air-bodies could never be observed.

The Case of Solitary Waves.—The behavior of a solitary wave with a height of $H = 15.5$ cm and a wave celerity of $\omega = 230$ cm per sec which broke in front and struck against the vertical wall of a breakwater-model, founded on a mound with a top of 20-cm width, a seaward slope of 1:10, and a height of $h_d = 37$ cm, was studied in a depth of $h_1 = 4.4$ cm by the use of a 16 mm high speed movie in order to compare it with the behavior of a breaking waves in shallow-water. The solitary wave produced the maximum pressure $p_{max} = 150$ gr per sq cm slightly above the still water surface by fully breaking just before the vertical wall.

Fig. 3 shows the state of the profile of a breaking solitary wave at various instants at intervals of 1/10 sec and $h_1 = 4.4$ cm, and $H = 15.5$ cm. At an instant when the water surface of the breaking solitary wave came to the location shown by a Ⓜ line in Fig. 3, the wave pressure indicated the maximum value p_{max}. The lines ②, ③, ④, ----, and ⑦ show the water surface corresponding to the times ②, ③, ④, ----, and ⑦ in Figs. 9 (a) and (b). The lines ⑧, ⑨, ⑩, and ⑪ show the descending state of the water surface. According to these figures, it is seen that the beginning of the pressure-action occurs later on the upper pressure-gauge than on the lower one; on the contrary, the peak pressure acts earlier on the upper gauge than on the lower one. Therefore it seemed that the masses of air might have been easily enclosed between the wave front and the vertical wall. Nevertheless, the photographs of the high speed movie could not confirm the enclosure of the masses of air as well as the compression of air bubbles entrained in the breaking wave.

Wave Cerelity and Water-Particle Velocity just before and after Striking against a Vertical Wall.

The Case of Shallow-water Waves.—The location of the water surface and of water particles in the breakers before and after shallow-water waves broke in front of and struck against the breakwater-model are shown in Fig. 2. In Fig. 2 the lines ①, ②, ----, and ⑦ show the water surfaces of a plunging breaker and the continued run-up waves at intervals of 1/10 sec and the lines ⑧, ⑨, ⑩, and ⑪ the descending wave surfaces at the same intervals. From Fig. 2 it was found that just before the breaking wave struck against the vertical wall, the wave celerity reached the maximum value, and the horizontal component of the maximum wave celerity reached 200 cm per sec which was slightly greater than $\omega = 182$ cm per sec at the horizontal bottom of the channel.

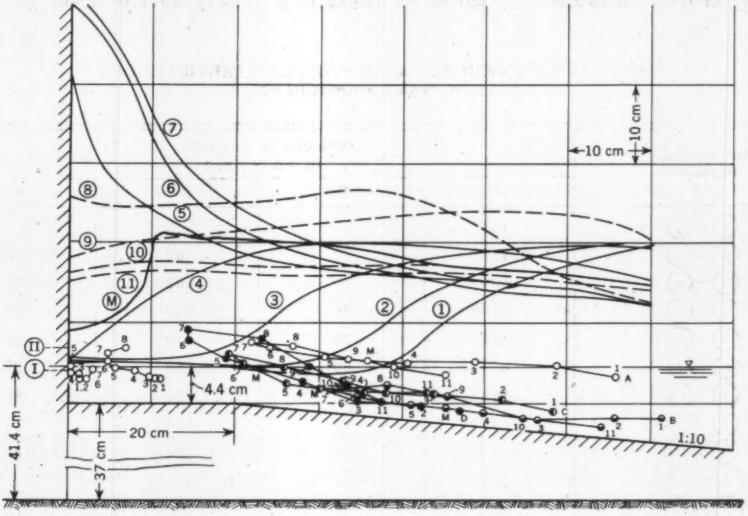


FIG. 3.—WATER SURFACE SHAPES OF A BREAKING WAVE, (SOLITARY WAVE)

The paths and velocities of the motion of water particles in the breaking waves were measured by the use of the high speed movie, by throwing into the breakers small spheres made of synthetic resin with diameters of 1.5 cm to 1.9 cm and specific weights of 1.00 to 1.12. The results are shown in Fig. 2, as well as in Table 2.

Table 2 indicates that the velocities of the synthetic resin particles reached the maximum value just before or after the breaking waves struck against the vertical wall. The diameters of the resin-particles were too large to rush up with the water of a thickness of several centimeters running up along the vertical wall. The path of motion of the A resin-particle shown in Fig. 2 appears to show an elliptic orbit of an underwater particle situated at the back side of the breaking wave. The velocity of the A particle from 1 to 2, $u_A = 36$ cm per sec can compare favorably with the theoretical maximum horizontal velocity of a water particle on an elliptic orbit at the location, $u_{max.} = 31$ cm per sec. But the velocities from 2 to 3 and from 3 to 4 are larger than the theoretical velocities of the water particle, probably because of the effect of mass-transport due to the wave breaking.

The Case of a Solitary Wave.—The behavior of the breaking of a solitary wave is comparatively different, as shown in Fig. 3, from that of a shallow-water wave, because the progressive nature of a solitary wave is never superimposed with a retrogressive wave reflected by a vertical wall, and the trough of a solitary wave exists always on or slightly above the still water surface.

TABLE 2.—OBSERVED VELOCITIES OF RESIN PARTICLES IN A BREAKING WAVE SHOWN IN FIG. 2

Location of Particle	Velocity of Particle, in cm per sec			
	A	B	C	D
①~②	36	48	8	...
②~③	46	62	62	70
③~④	50	80	102	80
④~⑤	38	76	120	86
⑤~⑥	42	76	70	80
⑥~⑦	24	60	50	58
⑦~⑧	16	30	24	42
⑧~⑨	18	20	8	14
⑨~⑩	...	50	24	10
⑩~⑪	...	78	...	...
⑪~⑫	...	78	...	...

From Fig. 3 it is known that just before striking against the vertical wall, the celerity of the solitary wave front reached the maximum value 220 to 230 cm per sec which coincided with the wave celerity before breaking, $\omega = 230$ cm per sec. The velocities of the synthetic resin particles at the various locations of the breaking solitary wave are presented in Table 3. As is seen in Fig. 3, the particles in front of the vertical wall indicate only a slight motion, and this is considered to be the result of no retrogressive wave reflected from the wall. Figs. 4 and 5 show the state of the breaking solitary wave shown in Fig. 3 at instants when the wave pressure began to act on the pressure-gauge and when the pressure reached the maximum value respectively.

Behavior of Breaking Waves at the Instant of Peak Pressure.

The Case of Shallow-Water Waves.—The time variation curves of breaking wave pressures produced from oscillograms are shown in Fig. 1 (a) and (b), in which the lines ①, ②, ③, corresponds to the water surfaces ①, ②, ③, shown in Fig. 2, respectively. The curve of Fig. 6 (a) is the record on a lower pressure-gauge located at 7.9 cm below SWL, and the curve of Fig. 6 (b) is the record on an upper pressure-gauge located at 5.3 cm below SWL.

From Figs. 2 and 6, it is known that the wave pressure begins to act at an instant when the breaking wave comes in contact with the pressure-gauge, and reaches the peak value at a very short interval of approximately 1/30 sec to 1/60 sec and drops down at a much shorter interval of 1/100 to 1/500 sec. Fig. 7 shows the state of a breaking wave in Fig. 2 at an instant when the wave pressure began to act on the pressure-gauge. Fig. 8 shows the state of a breaking wave at an instant when the pressure reached the maximum value for a shallow water wave in Fig. 2. From studies due to synchronizing the oscillograph connected to 3 pressure-gauges, with the high speed movie, it was found that in shallow-water waves the peak pressures at the various elevations of the vertical wall occurred in turn from the bottom of the wall to the highest point of wave run-up.

TABLE 3.—OBSERVED VELOCITIES OF RESIN PARTICLES IN A BREAKING SOLITARY WAVE SHOWN IN FIG. 3

Location of Particle	Time, in $\frac{1}{120}$ sec	Velocity of Particle, in cm per sec					
		A	B	C	D	E	F
①~②	12	70	60	65	45	10	0
②~③	13	92	85	83	46	7	6
③~④	12	85	70	80	70	18	10
④~⑤	16.5	73	62	69	69	20	1
⑤~⑥	14	60	56	52	43	12	17
⑥~⑦	8.5	51	42	28	21	17	17
⑦~⑧	26.5	25	23	32	41	10	...
⑧~⑨	18	47	57	47	47	31	...
⑨~⑩	17	42	67	39	35	...	...
⑩~⑪	24	33	50	28	25	...	...

The Case of a Solitary Wave.—Figs. 9 (a) and (b) show the time-variation curves of the breaker of a solitary wave recorded on a lower pressure-gauge located 0.6 cm below SWL and an upper gauge located 2.0 cm above SWL respectively. The peak pressures in Figs. 9 (a) and (b) are $p_{peak} = 95$ gr per sq cm and 89 gr per sq cm respectively, and the maximum pressure, $p_{max} = 150$ gr per sq cm, which occurred at 4.0 cm above SWL. According to Fig. 9, the wave pressure begins to act on the lower pressure-gauge 3/120 sec earlier than on the upper pressure-gauge. On the contrary, the peak pressure occurs on the upper gauge 1/120 sec earlier than on the lower gauge. From the re-

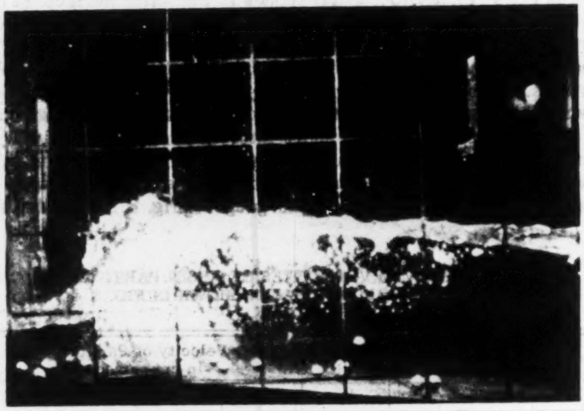


FIG. 4.—INSTANT WHEN WAVE PRESSURE BEGAN TO ACT

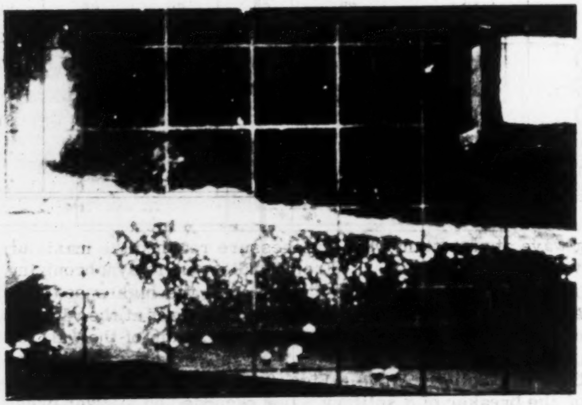
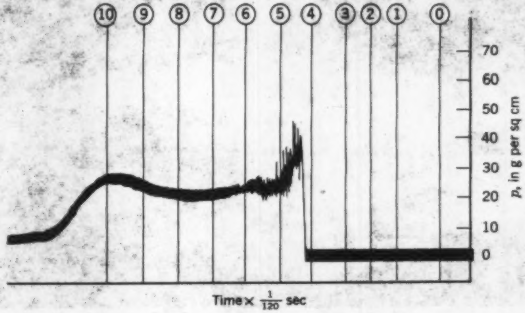
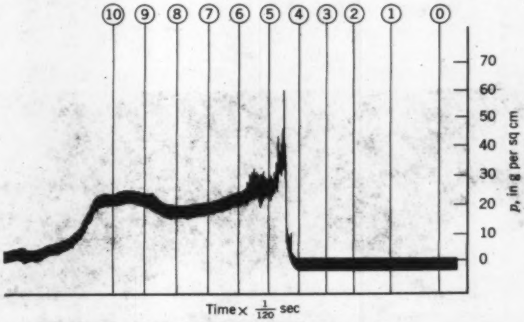


FIG. 5.—INSTANT WHEN PRESSURE INDICATED THE MAXIMUM VALUE

sults obtained in synchronism with the high speed movie and the oscillograph connected with three pressure-gauges, it was found that when a solitary wave broke in front of and struck against a vertical wall, the peak pressure occurred the earliest at a location in which the breaking wave-crest struck against the wall, and the peak pressure at the location was the maximum pressure on the wall. Since the crest of the breaking solitary wave struck at or slightly above SWL against the vertical wall, the maximum pressure occurred always at or slightly above SWL. For example, in Fig. 9 it occurred at 4 cm above SWL. The peak pressures at various heights above SWL did not decrease so quickly with increasing distance above SWL as those did in shallow water breaking waves, indicating $p_{peak} = 70$ to 100 gr per sq cm even at a distance of a wave-height H = 15.5 cm above SWL for the maximum pressure



(a) RECORD ON A LOWER GAGE



(b) RECORD ON AN UPPER GAGE

FIG. 6.—PRESSURE-TIME CURVES OF THE BREAKING WAVE OF SHALLOW WATER WAVE SHOWN IN FIG. 2

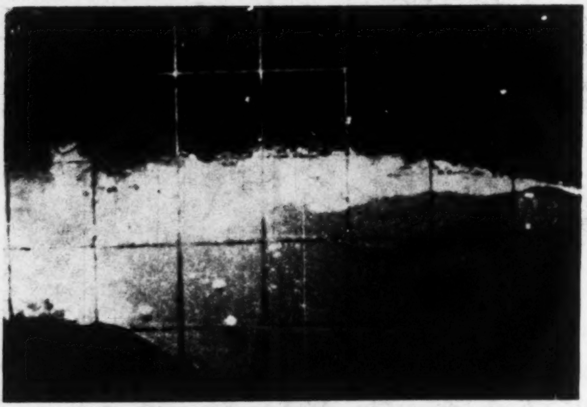


FIG. 7.—INSTANT WHEN WAVE PRESSURE BEGAN TO ACT

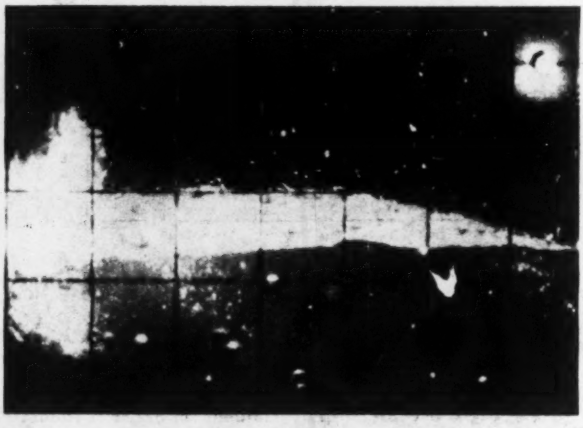
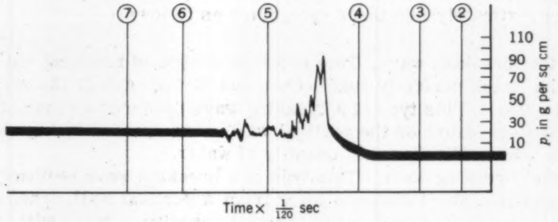


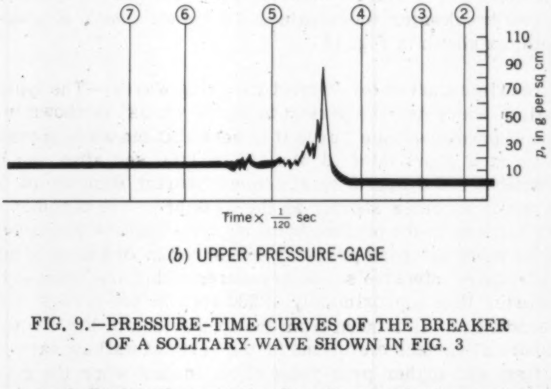
FIG. 8.—INSTANT WHEN PRESSURE INDICATED MAXIMUM VALUE

on the wall, p_{max} = 150 gr per sq cm. The facts above mentioned show that the behavior of a breaking solitary wave which strikes against a breakwater is distinguishably different from that in a shallow-water breaking wave.

Velocity of Water Running up along a Vertical Wall and Height of Spray.— When the breaker of a shallow-water wave approaches a composite-type breakwater, water particles near the trough of the breaker obtain an upward component of velocity by the effect of the slope of a base-mound. Owing to this, a submerged flow with opposite direction against a progressive wave cannot occur on the mound slope at an instant and just after the breaking wave strikes against the vertical wall. Therefore it is considered that nearly all of the water volume transported toward the breakwater by the breaking wave runs up with a thin layer of water along the vertical wall. The velocity of water running up along the wall reaches the maximum at the beginning of run-up. The maximum run-up velocity was measured by the use of the high speed movie camera and the oscillograph as $v_{max.} \doteqdot 4.1$ to 4.3 m per sec, which



(*a*) LOWER PRESSURE-GAGE



(*b*) UPPER PRESSURE-GAGE

FIG. 9.—PRESSURE-TIME CURVES OF THE BREAKER OF A SOLITARY WAVE SHOWN IN FIG. 3

agreed well with the computed values by the equation of continuity, using 5 cm as the thickness of the running up water-layer.

Assuming the initial velocity of run-up as $v_{max.} = 4.2$ m per sec which is an average value of the measured velocities, the height of the run-up or the spray is obtained: $\frac{v_{max}^2}{2g} = 0.90$ m. The value compares favorably with the measured height of wave run-up.

Pressure-Time Curves of Breaking Waves.

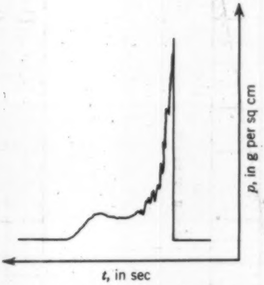
Kinds of Waves Striking Against Breakwaters.—The configuration of waves observed when breaking against breakwaters changes in a wide range, depending on the types of breakwaters, the characteristics of waves, and the ratio of a water depth in front of a vertical wall to a wave height, h_1/H, and can be divided approximately into three categories as follows:

a. Perfect breaking wave. This is a typical type of breaking wave observed when waves break perfectly just before and strike against the vertical walls of a breakwater. This type of a breaking wave produces an instantaneous and severe shock-pressure on the vertical wall, accompanied by high spray which contains a comparatively small quantity of water.

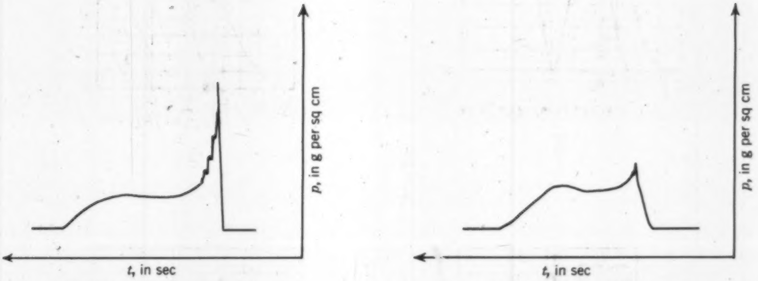
b. Partial breaking wave. This type of a breaking wave is observed when a wave breaks at a short distance away from a vertical wall, when only a part of a wave-crest breaks in front of the wall, or when a wave with the appearance of a non-breaking wave plunges into the wall, as if it were a jet flow. This type includes all kinds of breaking waves from the perfect breaking wave previously defined to a clapotis.

c. Clapotis. This means a standing wave, which is a kind of non-breaking wave. The condition in which a standing wave is produced in front of the vertical wall of a breakwater was obtained for the steepness of a wave from the experiments, as shown in Fig. 15.

Pressure-Time Curves for Perfect Breaking Waves.—The typical shape of pressure-time curve in "the perfect breaking waves" is shown in Fig. 10 (a). In this type of pressure-time curve it is seen that the wave pressure reaches a peak value at a short interval of 1/30 to 1/60 sec after the beginning of pressure-action, and drops down at a much shorter duration of $\tau = 1/100$ to 1/500 sec which becomes shorter as the peak pressure becomes greater. It sometimes happens in the perfect breaking wave that the peak pressure reach two times or more the peak pressure measured in ordinary breaking waves. Such extraordinary intensive shock-pressures which are supposed to act at an interval shorter than approximately 1/200 sec, do not always occur even at the same conditions. As is seen from the comparison of Fig. 2 with Fig. 6, each wave pressure at various elevations of the vertical wall appears to have indicated its first and higher peak value at an instant when the main part of a breaking wave struck against the elevation on the wall. Then the pressure decreases rapidly as the wave runs up along the wall, and drops down to the trough of the pressure-curve at approximately 1/10 sec after the wave has reached the highest point of the run-up. As the spray (the run-up wave) descends quickly along the wall, the pressure increases again and reaches the second and lower peak value at an instant when the spray stopped to descend. After this the pressure decreases slowly to the initial value, as the spray

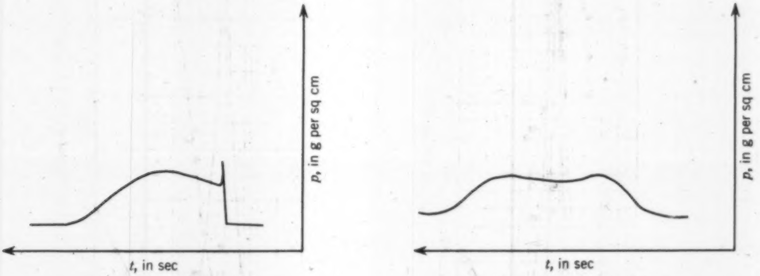


(*a*) PERFECT BREAKING WAVE



(*b*) PARTIAL BREAKING WAVE

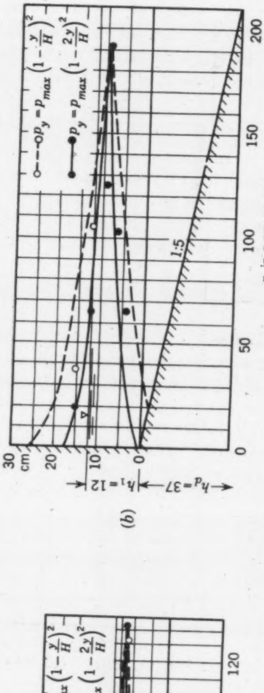
(*c*) PARTIAL BREAKING WAVE



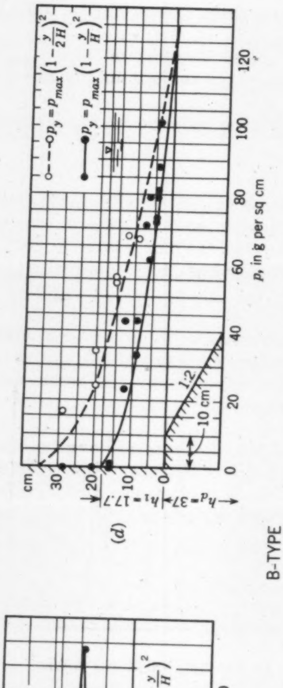
(*d*) PARTIAL BREAKING WAVE

(*e*) PARTIAL BREAKING WAVE

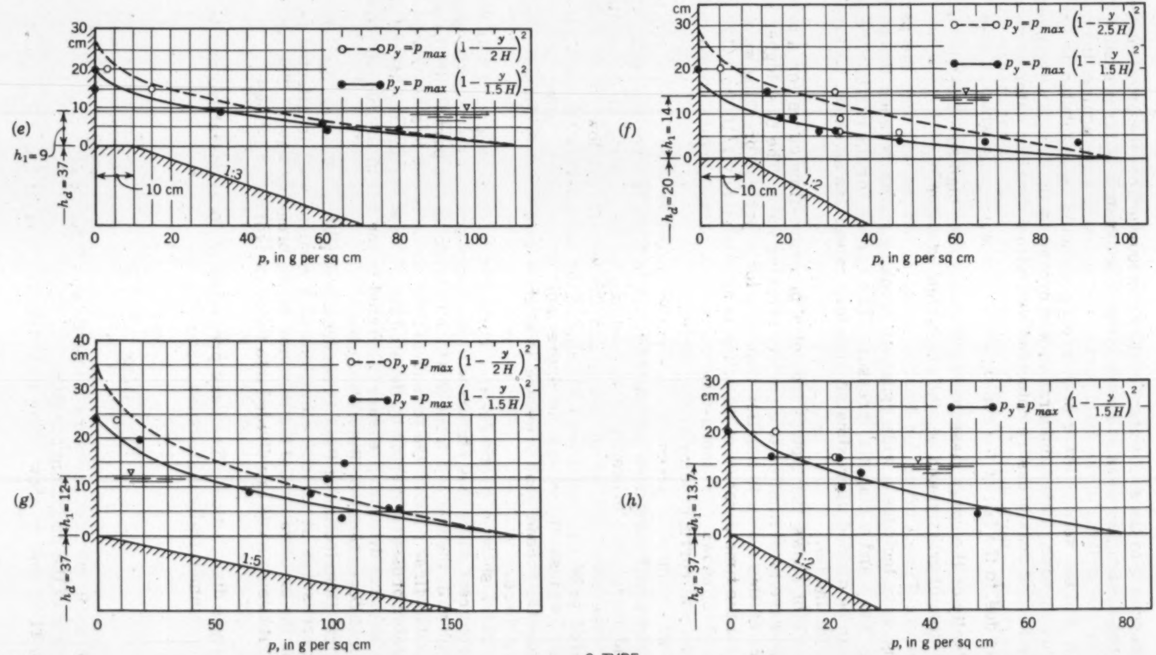
FIG. 10.—PRESSURE-TIME CURVES


(a)
(b)
$p_y = p_{max}\left(1-\frac{y}{H}\right)^2$
$p_y = p_{max}\left(1-\frac{2y}{H}\right)^2$
$h_1 = 13.7$
$h_d = 37$
$h_1 = 12$
cm
30
20
10
0
1:2
1:5
10 cm
0
20
40
50
60
80
100
120
150
200
p, in g per sq cm

A-TYPE


(c)
(d)
$p_y = p_{max}\left(1-\frac{2y}{H}\right)^2$
$p_y = p_{max}\left(1-\frac{y}{H}\right)^2$
$p_y = p_{max}\left(1-\frac{y}{2H}\right)^2$
$h_1 = 13.7$
$h_1 = 17.7$
$h_d = 37$
cm
30
20
10
0
1:2
10 cm
0
20
40
50
60
80
100
120
150
p, in g per sq cm

B-TYPE



C-TYPE

FIG. 11.—VERTICAL DISTRIBUTIONS OF PEAK PRESSURES AND MAXIMUM SIMULTANEOUS PRESSURES

slowly descends to the still water level. As is known from the prior interpretation, the first higher peak pressure is due to a dynamic water pressure generated by the striking of the main part of the breaking wave against the vertical wall, and the second lower peak pressure should be attributed to the fall of the run-up wave. This consideration was proved by the fact that the intensity of the second peak pressure coincided well with a dynamic water pressure-intensity due to the fall of the run-up wave, $p = w_o \frac{v^2}{2g}$, in which v was the fall-velocity of the spray measured by the use of the high speed movie, g the acceleration of gravity, and w_o the unit weight of water.

Pressure-Time Curves for Partial Breaking Waves.—Though the partial breaking wave includes a wide range of breaking waves between the perfect breaking wave and a clapotis, pressure-time curves for the partial breaking waves can be divided approximately into four categories as shown in Figs. 10 (b), (c), (d) and (e).

Fig. 10 (b) shows the typical type of a pressure-time curve for a partial breaking wave most similar to a perfect breaking wave. The first higher peak pressure of this curve is smaller than that for a perfect breaking wave, and in general the time duration of its action is approximately 1/100 to 1/120 sec.

Fig. 10 (c) shows a pressure-time curve for a partial breaking wave not so severe as that corresponding to Fig. 10 (b).

Fig. 10 (d) shows a pressure-time curve recorded when a wave with long period and small steepness partially breaks in front of a vertical wall. The first and sharp peak pressure is attributable to the striking of the partially broken wave-crest against the wall, and the second and low round peak is considered to be due to the weak striking of the main part of the wave against the wall, because there was no appreciable wave run-up along the wall. Therefore the second peak pressure is slightly greater than the static water pressure there.

Fig. 10 (e) shows a pressure-time curve recorded when a wave with comparatively great steepness strikes with such appearance as a clapotis against a vertical wall, the depth in front of which is large as compared with the wave height. The first peak of the pressure-time curve is due to a horizontal dynamic water pressure exerted by the striking of the wave, and the second peak due to a vertical dynamic pressure generated by the fall of the run-up wave.

Vertical Distribution of Wave Pressures.

Vertical Distribution of Peak Pressures over a Wall.—Vertical distribution of peak pressures at various elevations on a vertical wall is affected by the shape of the breakwater and the behavior of the breaking wave. The types of the distribution curves are divided into three categories as follows:

a. A-type distribution. The peak pressures at locations above and below SWL are symmetrically distributed, according to

$$p_y = p_{max.} \left(1 - \frac{y}{H}\right)^2, \quad \ldots\ldots\ldots\ldots\ldots \quad (4)$$

in which p_y is the peak pressure-intensity at a distance of y above and below the location of the maximum pressure.

Figs. 11 (a) and (b) show some experimental data of the A-type distribution.

b. B-type distribution. The maximum pressure occurs at the bottom of a vertical wall, and the peak pressures above the bottom decrease according to

$$p_y = p_{max.} \left(1 - \frac{y}{2\,H}\right)^2 , \quad \ldots\ldots\ldots\ldots\ldots \quad (5)$$

Some experimental data are shown in Figs. 11 (c) and (d). Fig. 11 (d) is a mixed type of A and B.

c. C-type distribution. The maximum pressure occurs at the bottom of a wall, and the peak pressures above the bottom decrease according to

$$p_y = p_{max.} \left(1 - \frac{y}{2\,H}\right)^2 , \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

or

$$p_y = p_{max.} \left(1 - \frac{y}{2.5\,H}\right)^2 , \quad \ldots\ldots\ldots\ldots\ldots \quad (7)$$

Several experimental data are shown in Figs. 11 (e), (f) (g) and (h) in which Eq. 6 or 7 is indicated by a broken line.

Vertical Distribution of the Maximum Simultaneous Pressures.—Owing to the films of the high speed movie and the oscillograms of pressure-time variation curves taken when the breakers of shallow-water waves struck against composite-type breakwaters, the peak pressures at various elevations of the vertical wall of the breakwaters did not occur at the same time. In general, there were slight retardations of occurrence in turns toward the wave crest from the bottom of the wall. Therefore the maximum resultant wave pressure inducing the sliding and overturning of the vertical wall is not equal to the resultant of the peak pressures, but equal to the resultant of the simultaneous wave pressures exerting at the same instant as the occurrence of $p_{max.}$ on the wall. A large number of experiments which had been carried out in this laboratory proved that the vertical distribution of wave pressures acting simultaneously with $p_{max.}$ on various elevations of the vertical wall indicated similar trends to those of peak pressure and could be divided into three categories as will be shown in the following paragraphs. But the maximum resultant of the simultaneous wave pressures was relatively smaller as compared with the resultant of peak pressures.

a. A-type distribution. The maximum pressure on the wall, $p_{max.}$, occurs at or in the vicinity of the still water surface, and pressures acting simultaneously with $p_{max.}$ above and below SWL decrease more quickly than in peak pressures, according to the following equation

$$p_y = p_{max.} \left(1 - \frac{2y}{H}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

in which p_y is the simultaneous pressure-intensity at a distance of y above and below the location of $p_{max.}$. Figs. 11 (a) and (b) show the experimental examples of A-type distribution, in which the distribution curves of the maximum simultaneous pressures are shown by a full line, and the experimental data by double circle.

The resultant, P, of the maximum simultaneous wave pressures acting on the unit longitudinal length of a vertical wall is computed for the case of A-type distribution by the equation

$$P = 2 \int_0^{H/2} p_y \cdot dy = 2\, p_{max.} \int_0^{H/2} \left(1 - \frac{2y}{H}\right)^2 \cdot dy = \frac{1}{3} p_{max.} H \quad \ldots (9)$$

Though the conditions necessary for the production of A-type pressure distribution are difficult to decide precisely, they seem to be defined as follows: when waves with short periods of $T \leqq 1.5$ sec and large steepnesses of $H/L \geqq 0.045$ have broken at shallow water depths ($h_1/H \leqq 1.0$) just in front of and have struck against the vertical wall of composite-type breakwaters which have high base-mounds ($S/L \geqq 0.27$) with a 1:2 front slope and the horizontal tops of considerably large width or with a 1:5 front slope, the wave crests attack against the wall at or in the vicinity of the still water surface, exerting generally large shock pressures and sometimes extraordinary severe shock-pressures. The maximum simultaneous pressures indicate the vertical distribution of the A-type curve.

TABLE 4.—SUMMARY OF A-TYPE DISTRIBUTION

Breakwater				T, in sec	h_1, in cm	h_1/h_o	h_1/H	H/L	Simultaneous Pressure Distribution
h_d, in cm	B, in cm	Front Slope 1:m	S/L						
37	10	1:2	≧ 0.34	1.29 ~1.31	6.4 ~13.7	0.147 ~0.270	0.53 ~0.91	0.056 ~0.065	A-type
37	10	1:2	≧ 0.30	1.42	19.7 ~21.7	0.348 ~0.369	1.0 ~1.1	0.071 ~0.073	A-type
37	0	1:5	≧ 0.74	1.29	6.0 ~15.0	0.140 ~0.288	0.42 ~0.69	0.064 ~0.086	A-type
37	10	1:2	0.29	1.5	13.7 ~17.7	0.270 ~0.324	0.79 ~0.96	0.061 ~0.065	A, B Mixed type
37	10	1:2	≧ 0.31	1.5	≦ 6.4	0 ~0.147	0 ~0.53	0.045	A-type

Table 4 is a summary of the A-type distribution.

b. B-type distribution. The maximum pressure occurs at the bottom of a vertical wall, and pressures above the bottom decrease with increasing elevation in accordance with the following quardratic equation

$$p_y = p_{max.} \left(1 - \frac{y}{H}\right)^2, \quad \ldots\ldots\ldots\ldots\ldots (10)$$

in which p_y represents the pressure-intensity acting simultaneously with $p_{max.}$ at an elevation of y above the bottom of a vertical wall. The experimental examples of B-type distribution are shown in Figs. 11 (c) and (d).

The resultant, P, of the maximum simultaneous pressures on the unit longitudinal length of a vertical wall is obtained by

$$P = \int_0^H p_y \quad dy = p_{max.} \int_0^H \left(1 - \frac{y}{H}\right)^2 \quad dy = \frac{1}{3}\, p_{max.}\; H . \ldots \ldots (11)$$

This is the same as Eq. 9.

From Figs. 11 (c) and (d), the B-type distribution appears to occur when waves with large steepnesses of $H/L \geqq 0.050$ and a short period of $T \doteqdot 1.3$ sec have broken at large depths of $h_1/H \doteqdot 1.0$ before reaching the vertical wall and plunged into the bottom of the wall, or have struck as a spilling breaker against the vertical wall of a breakwater which has a high base-mound ($S/L \geqq 0.27$) with considerably wide top, and when waves with moderate periods of $T \doteqdot 1.5$ to 1.6 sec have plunged into the bottom of the vertical wall at moderate depths of $h_1/H \doteqdot 0.6$ to 1.3. But it is difficult to decide precisely the conditions necessary for the production of the B-type distribution, and the B-type distribution is supposed to occur at a transition state from A-type distribution to C-type. The B-type distribution was observed only in a few cases.

c. C-type distribution. The maximum pressure, $p_{max.}$, in C-type distribution also occurs at the bottom of a vertical wall such as B-type distribution, but run-up along the wall is larger than that of B-type, according to

$$p_y = p_{max.} \left(1 - \frac{y}{1.5\,H}\right)^2 , \ldots \ldots \ldots \ldots \ldots (12)$$

The experimental examples of the C-type distribution are shown in Figs. 11 (e), (f), (g) and (h).

The resultant, P, of the maximum simultaneous wave pressures on the unit longitudinal length of a breakwater is obtained for the C-type distribution by the equation

$$P = \int_0^{1.5\,H} p_y \quad dy = p_{max.} \int_0^{1.5\,H} \left(1 - \frac{y}{1.5\,H}\right)^2 \quad dy = \frac{1}{2}\, p_{max.}\; H \ldots (13)$$

In general the C-type distribution occurs when a wave with large steepness breaks as a spilling breaker at a large water depth of h_1 as compared with the height, h_d, of base-mound, and plunges into the bottom of the vertical wall of a breakwater. Therefore the run-up along the wall is larger than that of B-type.

The conditions in which the C-type distribution occurs are as follows:

(1) At a composite breakwater with the horizontal top of the base-mound When the front slope of the mound is 1:2 and S/L is smaller than 0.27, or the front slope is 1:3 and S/L is equal to or smaller than 0.50.

(2) At a composite breakwater without the horizontal top of the base-mound From all cases when the front slope of the mound is 1:2 to 1:3, and for most of the cases when the front slope of the mound is 1:5.

(3) At a composite breakwater with low base-mound ($h_d/L < 0.12$). For all cases when the front slope of the mound is 1:2, 1:3, and 1:5, no matter what the width B of the top of the mound may be.

(4) Vertical distribution of the maximum simultaneous pressures by breaking waves with a period of 1.9 sec. Since the steepnesses of the waves with a period of 1.9 sec and heights of H = 6.8 cm to 13.2 cm were as small as H/L = 0.019 to 0.033, these waves collapsed weakly in front of the vertical wall of a composite breakwater, the base-mound of which has a 1:2 front slope, a 10 cm-wide top, and a height of 37 cm, acting on the wall considerably smaller pressures than the shock pressures of other breaking waves with short periods of T < 1.9 sec. The vertical distribution of the maximum simultaneous pressures over the wall were also quite different such as the maximum pressure, $p_{max.}$, in each case when h_1 changed from 4.4 cm to 13.7 cm, occurred at the constant elevation of 6 cm above the top of the base-mound and pressures acting simultaneously with $p_{max.}$ decreased above $p_{max.}$ in accordance with the rectilinear Eq. 14 and below $p_{max.}$ with the rectilinear Eq. 15, as shown in Figs. 12 (a) and (b) for $h_1 \leqq 6.4$ cm,

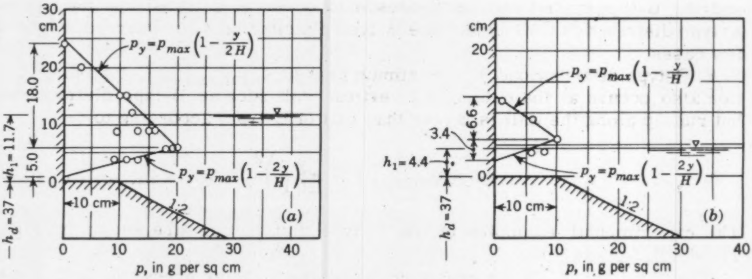


FIG. 12.—VERTICAL DISTRIBUTIONS OF MAXIMUM SIMULTANEOUS PRESSURES

$$p_y = p_{max.}\left(1 - \frac{y}{H}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14a)$$

for $h_1 = 8.4 \sim 13.7$ cm

$$p_y = p_{max.}\left(1 - \frac{y}{2H}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14b)$$

and for $h_1 = 4.4 \sim 20.0$ cm,

$$p_y = p_{max.}\left(1 - \frac{2y}{H}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

When the depth, h_1, in front of the vertical wall reached 24 cm, $h_1/H = 1.82$, H/L = 0.033, the intensities and vertical distribution of the pressures became nearly the same as those of clapotis.

Vertical Distribution of the Peak Pressures of a Breaking Solitary Wave over a Wall.—When a solitary wave broke completely at the depths of $h_1 = 4.4$ cm and 6.4 cm just in front of the vertical wall of a composite breakwater which had a base-mound with a top of 20 cm width and a gentle front slope of 1:10, the peak pressures at various elevations of the wall distributed as shown in Figs. 13 (a) and (b). In Figs. 13 the vertical distributions of the peak pressures over the wall by a breaking solitary wave are considerably different from those by breaking shallow-water waves, such that the peak pressures did not decrease so quickly as in the breakers of shallow-water waves, indicating peak pressures as large as 60% to 90% of $p_{max.}$ even on an elevation of $y = 2H$ from the bottom of the wall, and producing run-ups as high as 3H.

Theoretical Considerations for the Shock Pressures of Breaking Waves.

Momentum Equation.—The most important reason for the occurrence of the high shock pressures of breaking waves is considered to be an instantaneous momentum-change due to the striking of breaking waves against a wall. That is, the high shock pressures are due to the fact that the momentum-transport of breaking-waves is not made so continuously as that of jet flows, but abruptly done, and the transported momentum is instantaneously converted into impulse by striking against a wall.

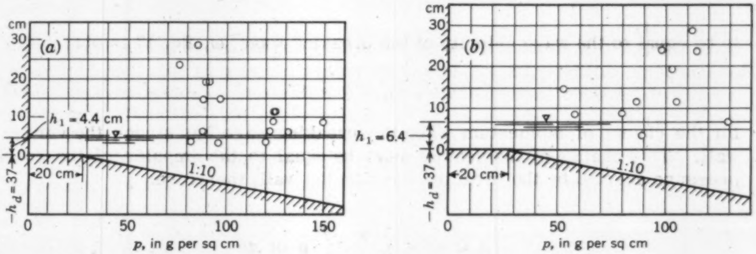


FIG. 13.—VERTICAL DISTRIBUTION OF PEAK PRESSURES OF A BREAKING SOLITARY WAVE

The time-integral of shock pressures during a period exerted by breaking waves on the unit longitudinal length of a vertical wall is

$$I = \int_0^{h_2} \int_0^{\tau} p \, dt \, dh \, , \ldots\ldots\ldots\ldots\ldots \quad (16)$$

in which h_2 is the height from the bottom of a wall to the wave-crest, and τ represents the duration of the shock pressures of breaking waves.

The volume of water transported by a breaking wave during a period on the unit traverse length of the wave-crest is expressed by Q, the momentum transported by the breaking wave during a period is $\rho Q u_b$, in which u_b is the horizontal water-particle velocity in the breaking wave and is assumed to be constant over all points in the wave. Since the horizontal water-particle velocity u_b at the instant of striking a wall has been proved by the experiments

TABLE 5.—MOMENTUM

Characteristics of Wave			h_1, in cm	h_o, in cm	Q, in cu cm	ω, in cm per sec
H, in cm	L, in cm	T, in sec				
8.5	300	1.6	4.6	42.6	1880	187
11.7	200	1.2	5.6	42.6	2080	170
12.7	272	1.5	6.4	43.4	2350	179
12.8	271	1.5	8.4	45.4	2520	176
14.0	282	1.5	11.7	48.7	2930	187
14.3	315	1.6	17.7	54.7	3520	195
17.2	258	1.4	13.7	50.7	3460	187
16.3	308	1.6	19.7	56.7	3986	200
16.1	319	1.6	21.7	58.7	4135	207

to be equal to the wave celerity of the breaking wave, ω, Eq. 17 is obtained.

$$\rho \; Q \; u_b = \rho \; Q \; \omega \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

for the change of momentum when the breaking wave has struck the vertical wall. This change of momentum must be equal to the impulse of the shock pressure exerted by the breaking wave on the wall, therefore:

$$\rho \; Q \; \omega = \int_0^{h_2} \int_0^{\tau} p \; dt \; dh \quad \ldots\ldots\ldots\ldots\ldots (18)$$

If Q is assumed to be approximately equal to the volume of a solitary wave

$$Q = 4 \, h_o^2 \sqrt{\frac{H}{3 \, h_o}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

in which h_o is the water depth, and H represents the height of a solitary wave.

Interpreting Eq. 18 more specifically, since the first higher and steeper peak pressure of a pressure-time curve produced by a breaking wave is due to a horizontal hydrodynamic pressure caused by the strike of the breaking wave against a vertical wall, and the second, much lower and more gentle peak due to a vertical hydrodynamic pressure caused by the fall of the run-up water of the breaking wave, the net momentum-change of the breaking wave to be transformed into the impulse of the shock pressure during the impact must be equal to the impulse defined by the area of the pressure-time curve under the first peak pressure. The impulse defined by the area of the pressure-time curve under the second peak may probably be transformed into the momentum-change of the retrogressive wave. If it is assumed that this impulse of the second peak pressure is equal to the total momentum which the retrogressive wave has gained, Eq. 18 may be obtained correctly as the momentum transport of the breaking wave is in the reverse direction to the momentum of the retrogressive wave.

For the impulses which have been exerted by perfect as well as partial breaking waves on the vertical walls which were constructed on a 1:5 slope

AND IMPULSE

$\rho\, Q\, \omega$, in gr cm per sec x 10^3	$\iint p\, dt\, dh$, in gr cm per sec x 10^3	$\dfrac{\iint p\, dt\, dh}{\rho\, Q\, \omega}$	Base-mound
351	334	0.95	B=0, 1:5
354	327	0.92	B=0, 1:5
421	406	0.96	B=10 cm , 1:2
443	443	1.00	B=10 cm , 1:2
548	595	1.08	B=10 cm , 1:2
686	907	1.32	B=10 cm , 1:2
647	626	0.97	B=10 cm , 1:2
797	598	0.75	B=10 cm , 1:2
855	723	0.85	B=10 cm , 1:2

and located on the top of 10 cm width of a base-mound with a 1:2 front slope, the left side of Eq. 18 was calculated by use of Eq. 19, and the right side of Eq. 18 was numerically integrated from the oscillograms obtained by the experiments. Data for nine experimental waves are given in Table 5. From Table 5 the ratios of the impulses to the momentum changes for the six waves are seen to be smaller than one. This may be due to the fact that a part of the momentum is converted to turbulence and into the vertical motion of the spray.

Derivation of an Equation for Shock Pressure.—Expressing the mass transport per unit area of a breaking wave during a duration τ of the first peak shock pressure of a pressure-time curve by ρ l, Eq. 18 yields

$$\int_0^{\tau} p\ dt = \rho\ l\ \omega, \quad \dots\dots\dots\dots\dots\dots \quad (20)$$

in which l is an assumed length of a water column transported during τ. A value of 3 cm to 5 cm has been obtained for l from the experiments. Putting the left side of Eq. 20 approximately

$$\int_0^{\tau} p\ dt = k\ p_{peak}\ \tau \quad \dots\dots\dots\dots\dots\dots \quad (21)$$

Eq. 20 yields

$$p_{peak} = \frac{\rho}{k}\ \frac{l\,\omega}{\tau} \quad \dots\dots\dots\dots\dots\dots \quad (22)$$

for gravity unit

$$p_{peak} = \frac{w_o}{k\,g}\ \frac{l\,\omega}{\tau} \quad \dots\dots\dots\dots\dots\dots \quad (23)$$

in which g is the acceleration of gravity, and w_o is equal to ρ g.

As previously mentioned, since the duration τ of the first peak pressure, p_{peak}, of the pressure-time curve of a breaking wave is very short (such as 1/100 sec to 1/500 sec) it may be assumed that the pressure-time curve of p_{peak} is approximately triangular, that is, $k = 1/2$. Hence from Eq. 23

$$p_{peak} = 2 \frac{w_o}{g} \frac{l\,\omega}{\tau} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

This is the basic equation for the shock pressures of breaking waves, from which formulas for the maximum shock pressures of ordinary and extraordinary breaking waves have been derived and are presented here. If the shock pressures (the first peak pressures) were caused by a water-hammer phenomenon, the theoretical peak values should be of the order of 10 to 100 times the peak pressures obtained from the experiments, and the theoretical duration of the peak pressures should be of the order of 1/100 the duration observed in the experiments. Therefore it is correctly considered that there does not exist any water-hammer phenomenon at an instant when breaking waves strike breakwaters, and only in this respect does the author agree with Bagnold's consideration.[4]

If it is assumed for comparatively weak partial breaking waves that horizontal water-particle velocities at the wave-crest are constantly equal to ω during the duration of the peak pressures, which are generally $\tau \doteq 1/80$ to 1/100 sec, the assumed length l can be written as

$$l = \tau\,\omega, \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (25)$$

Hence Eq. 24 yields

$$p_{peak} = 2 \frac{w_o}{g} \omega^2. \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (26)$$

This means that the peak pressure in a comparatively weak partial breaking wave is equal to 2 times the hydrodynamic pressure of a jet flow on a wall perpendicular to the jet. A comparative number of peak pressures corresponding to the p_{peak} of Eq. 26 were observed in the experiments of weak partial breaking waves.

Formula for the Maximum Shock Pressure.

Derivation of a Formula for the Maximum Shock Pressure.—Since the peak pressure in a pressure-time curve of a breaking wave at a point on a vertical wall is expressed analytically by Eq. 23, the maximum value of the peak pressures at various elevations on a vertical wall must also be expressed by the same equation as Eq. 23, in which l and τ cannot be obtained analytically except for a comparatively weak partial breaking wave, as previously mentioned. Therefore it is necessary that they be replaced with other factors easily measurable.

Main factors with which the value of $p_{max.}$ is affected are considered as follows:

a. Ratios of a depth h_1 in front of a vertical wall to a depth h_o over a part of horizontal bottom and a wave-height H therein, that is, h_1/h_o and h_1/H.

b. Width of the top of a base-mound, B, and its height h_d.
c. Wave-steepness H/L.

Since l and τ should be replaced with these various factors, from a dimension analysis

$$l = k_1 \quad h_1 \frac{h_1}{h_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (27a)$$

and

$$\tau = k_2 \quad \frac{L}{\omega} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (27b)$$

in which k_1 and k_2 are non-dimensional constants. Substituting Eq. 27 into Eq. 24 results in

$$p_{max.} = 2 \frac{w_o}{g} \frac{k_1}{k_2} h_1 \frac{h_1}{h_o} \frac{\omega^2}{L} \quad \ldots\ldots\ldots\ldots \quad (28)$$

Equating ω approximately to the wave celerity of the breaker of a solitary wave,

$$\omega = \sqrt{2.28 \ g \ H_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (29a)$$

For such waves of great steepness as $H/L \doteq 0.035$ to 0.070, the height H_b of the breaking waves is approximately equal to the height H of the non-breaking shallow water waves on a horizontal bottom basin, that is, $H_b \doteq H$, hence

$$\omega = \sqrt{2.28 \ g \ H} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (29b)$$

Substituting Eq. 29b into Eq. 28,

$$p_{max.} = 4.56 \frac{k_1}{k_2} w_o \ h_1 \frac{h_1}{h_o} \frac{H}{L} \quad \ldots\ldots\ldots\ldots \quad (30a)$$

If $4.56 \dfrac{k_1}{k_2} = K$, a dimensionless constant, Eq. 30a can be written as

$$p_{max.} = K \ w_o \ h_1 \frac{h_1}{h_o} \frac{H}{L} \quad \ldots\ldots\ldots\ldots \quad (30b)$$

Eq. 30b is the basic equation for a practical use of computing the maximum shock pressures exerted by breaking waves on vertical walls, and in this equation the main factors defined in a, b and c are included.

Evaluation of the Constant of Eq. 30b by Experimental Data.—Eq. 30b was checked with a number of $p_{max.}$ obtained from some 2,000 experimental data of shock pressures exerted by breaking waves on composite-type breakwaters. These experiments include various shapes of composite-type breakwaters such as the heights of the base-mounds h_d = 20, 33.4, 36, 37, and 38 cm, the front slopes = 1:2, 1:3, and 1:5, the top widths B = 10 cm and 0, and such characteristics of waves as heights $H \doteqdot 6$ to 24 cm, periods $T \doteqdot 1.2$ to 1.9 sec, and steepnesses $H/L \doteqdot 0.019$ to 0.083.

a. Formula for the maximum shock pressure of ordinary breaking waves. The relationship between the maximum pressure $p_{max.}$ and $h_1 \frac{h_1}{h_o} \frac{H}{L}$ of ordinary breaking waves, which include all of perfect and partial breaking waves except breaking waves acting extraordinary high shock pressures, are plotted in Fig. 14.

From Fig. 14 it is seen that the value of $p_{max.}$ increases linearly up to 130 gr per sq cm as the value of $h_1 \frac{h_1}{h_o} \frac{H}{L}$ increases in the region of $h_1 \frac{h_1}{h_o} \frac{H}{L} \leqq 0.22$ cm. But in the region of $h_1 \frac{h_1}{h_o} \frac{H}{L} > 0.22$ cm, $p_{max.}$ is nearly constant 100 to 130 gr per sq cm for increase of $h_1 \frac{h_1}{h_o} \frac{H}{L}$. This is considered to be due to the fact that as the depth h_1 in front of a vertical wall and the steepness H/L of a breaking wave increase up to a certain limiting value of $h_1 \frac{h_1}{h_o} \frac{H}{L}$, the breaking wave strikes more forcefully against the wall, but above the limit of $h_1 \frac{h_1}{h_o} \frac{H}{L}$ a wave has difficulty in breaking, resulting in no increase of the pressure. The limit of $h_1 \frac{h_1}{h_o} \frac{H}{L}$ may be 0.22 cm in the exment. As will be mentioned later, when h_1 increases and reaches $h_1/H \geqq 1.6$ to 2.0, a wave in front of a vertical wall transforms into a clapotis, the pressure of which approaches hydrostatic pressure.

In the region of $h_1 \frac{h_1}{h_o} \frac{H}{L} \leqq 0.22$ cm the maximum shock pressure of breaking waves, $p_{max.}$, in gr per sq cm

$$p_{max.} = 20 + 500\ w_o\ h_1 \frac{h_1}{h_o} \frac{H}{L} \quad \ldots\ldots\ldots\ldots \quad (31)$$

The relationship between $p_{max.}$ and $h_1 \frac{h_1}{h_o} \frac{H}{L}$ mentioned previously, is applicable for all kinds of shock pressure with the simultaneous distribution of A, B and C-types exerted by ordinary breaking waves on composite-type breakwaters which have the front slopes 1:2, 1:3, and 1:5, the top-widths $B = 10$ cm and $B = 0$, and the heights $h_d = 37$ cm and 20 cm of the base-mounds. But it is not applicable for low pressures by very weak breaking waves.

b. Formula for the maximum shock pressures of extraordinary breaking waves. The maximum shock pressures of extraordinary breaking waves reached generally up to two times or more those of ordinary breaking waves. The relationship between $p_{max.}$ and $h_1 \frac{h_1}{h_o} \frac{H}{L}$ for extraordinary breaking waves is plotted in Fig. 14. According to Fig. 14, although the relationship does not have a good correlation, the average values of $p_{max.}$ are related to $h_1 \frac{h_1}{h_o} \frac{H}{L}$ by the following equation, obtained by least squares analysis over the whole pressures,

$$p_{max.} = 280 \left(0.040 + h_1 \frac{h_1}{h_o} \frac{H}{L}\right)^{1/3} \quad \text{. . . . . (32)}$$

The upper limiting values of $p_{max.}$ are expressed by

$$p_{max.} = 300 \left(0.051 + h_1 \frac{h_1}{h_o} \frac{H}{L}\right)^{1/3} \quad \text{. . . . . (33)}$$

The values of $p_{max.}$ in extraordinary breaking waves are not always constant for the same condition concerning wave characteristics as well as breakwater-types, and change to a comparatively large extent with a slight variation of boundary conditions. The scatter of the experimental data in Fig. 14 may be considered to be partly due to this fact.

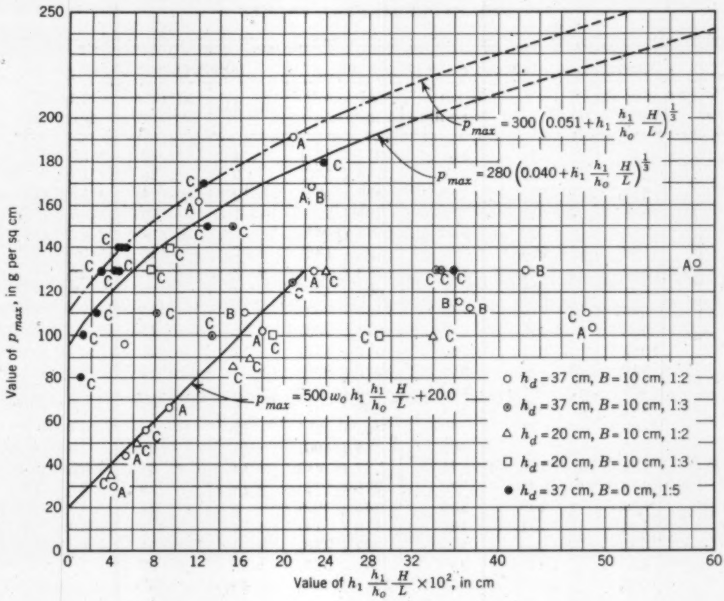


FIG. 14.—RELATIONSHIPS BETWEEN $p_{max.}$, AND $h_1 \frac{h_1}{h_o} \frac{H}{L}$

Comparisons of Values Computed by Eqs. 31, 32, and 33, as well as Minikin's[6] *and Hiroi's Formulas with Pressures Obtained by the Experiments.*—As mentioned previously, since there exists no generally accepted shock-pressure formula established on the basis of a sufficient number of prototype or experimental data, Hiroi's and Minikin's formulas were selected as the representatives of shock-pressure formulas for the comparison of the computed values of pressure with the experimental values of shock pressure.

For Ordinary Breaking Waves.

a. Comparison of the maximum shock pressure intensity. For the maximum pressure intensity $p_{max.}$ and the resultant P of the shock pressures exerted simultaneously with $p_{max.}$ by ordinary breaking waves on unit longitudinal length of the vertical walls of composite breakwaters, the comparisons of computed values by Eq. 31, Hiroi's and Minikin's formulas with measured pressures by the experiments were made and tabulated in Tables 6 and 7. In Table 6 Runs 1 to 23 are data for composite breakwaters with the top-width B = 10 cm of the base-mounds, and Runs 24 to 29 are data for weak breaking waves on composite breakwaters, the vertical walls of which are located on 1:2 slopes without the top-width. Therefore Runs 24 to 29 are originally exceptional for Eq. 31, but presented here only as a reference.

TABLE 6.—COMPARISONS FOR RESULTANT PRESSURES P PER UNIT LENGTH OF A WALL BETWEEN THE COMPUTED AND THE EXPERIMENTAL VALUES

Run No.	Resultant Pressure, P, in gr per cm				Percentage Deviation		
	Experimental value	Eq. 31	Hiroi's formula	Minikin's formula[a]	Eq. 31	Hiroi's formula	Minikin's formula[a]
1	185	193	312	237 (177)	+ 4	+ 69	+ 28 (- 4)
2	304	304	381	344 (262)	0	+ 25	+ 13 (- 14)
3	510	560	488	597 (480)	+ 10	- 4	+ 17 (- 6)
4	655	650	570	691 (560)	- 1	- 19	+ 6 (- 15)
5	780	780	750	1262 (1062)	0	- 4	+ 62 (+ 36)
6	120	164	295	192 (136)	+ 37	+ 146	+ 60 (+ 13)
7	230	234	350	273 (200)	+ 2	+ 53	+ 18 (- 13)
8	594	551	521	603 (475)	- 7	- 12	+ 2 (- 24)
9	690	710	776	1168 (963)	+ 3	+ 13	+ 70 (+ 40)
10	598	625	713	976 (783)	+ 5	+ 19	+ 63 (+ 31)
11	1045	925	760	1039 (832)	- 11	- 27	- 0.6 (- 20)
12	940	984	885	1239 (998)	+ 5	- 6	+ 32 (+ 6)
13	200	204	192	123 (91)	+ 2	- 4	- 39 (- 55)
14	451	504	304	292 (231)	+ 12	- 33	- 35 (- 49)
15	695	615	352	403 (325)	- 12	- 49	- 42 (- 53)
16	560	645	408	494 (400)	+ 15	- 27	- 12 (- 29)
17	105	120	144	67 (44)	+ 14	+ 37	- 36 (- 58)
18	378	441	286	238 (179)	+ 17	- 24	- 37 (- 53)
19	423	409	336	346 (269)	- 3	- 21	- 18 (- 36)
20	184	191	167	110 (74)	+ 4	- 9	- 40 (- 60)
21	465	530	270	287 (220)	+ 14	- 42	- 38 (- 53)
22	530	610	330	447 (360)	+ 15	- 38	- 16 (- 32)
23	1240	1010	844	1234 (1044)	- 19	- 32	- 0.5 (- 16)
24	250	206	428	164 (117)	- 18	+ 71	- 35 (- 53)
25	390	318	460	249 (186)	- 18	+ 18	- 36 (- 52)
26	486	459	502	379 (296)	- 6	+ 3	- 22 (- 39)
27	622	740	615	707 (576)	+ 19	- 1	+ 14 (- 7)
28	668	960	617	834 (685)	+ 44	- 8	+ 25 (+ 3)
29	850	1087	810	1346 (1134)	+ 28	- 5	+ 53 (+ 33)

[a] Figures in parenthesis are values substructed the static pressure from the total pressure.

According to these comparisons, the values of $p_{max.}$ computed by Hiroi's formula are all too small to compare with the experimental values, but the values of $p_{max.}$ by Minikin's formula are comparatively similar to the experimental values, indicating the maximum deviation of approximately ±60%, and especially smaller deviation for the cases of A-type distribution of the maximum simultaneous pressures.

b. Comparison of the resultant of the maximum simultaneous pressures. For the resultant P of the shock pressures exerted simultaneously with $p_{max.}$ on unit longitudinal length of a vertical wall, the comparisons of the computed values by the previously described three formulas with the experimental values are tabulated in Table 6.

In Table 6, P computed by Hiroi's formula are the values for composite breakwaters, the top of which are constructed 10 cm (this corresponds to 2.0 meters in prototype) high above the still water level. As is seen from Table 6, as far as the resultant pressure P concerns, the values computed by Hiroi's formula are comparatively similar to the experimental values in the nearly same order as the values by Minikin's formula, the deviation being in general within approximately ±70% except only one case of the maximum 146% deviation. This fact may be attributable to the reason that the method of calculation of P by Hiroi's formula was deviced from the experiences which had been obtained in Japan during some three decades for such composite breakwaters as the tops have been constructed at the elevations of 2m to 2.5m or less above the high water level of ordinary spring tide.

From these comparisons it was proved that both the values of $p_{max.}$ computed by Eq. 31 and of P by the use of Eqs. 9, 11, 13 and 31 agreed with the experimental results better than other two formulas, indicating approximately within ±20% of deviation except Runs 28 and 29 which were presented only for a reference.

For Extraordinary Breaking Waves.—For the much higher shock pressures of extraordinary breaking waves the comparisons of the computed values of $p_{max.}$ by Eq. 32 as well as by Hiroi's and Minikin's formulas with the experimental values are shown in Table 7.

As seen from Table 7, the values $p_{max.}$ computed by Hiroi's and Minikin's formulas are all too small to compare with the experimental values. The values $p_{max.}$ computed by Eq. 32 give a better agreement with the experimental values, indicating the maximum deviation of 30% and for most cases small deviations of within ±10%.

Application of Experimental Results to Prototype.

Scale Ratios of Model to Prototype.—Since all measurements of wave pressures and wave characteristics were carried out at the center longitudinal section of the 2-m width of the channel, and the lengths of the front slopes of the base-mounds of composite breakwaters were generally small compared with the wave-lengths used in the experiments of shallow-water waves, the energy loss of waves caused by frictional resistance at the bottom as well as both sides of the channel was small. Therefore Froude's law of similarity was applicable to these experiments with sufficient accuracy.

If the ratio of length l or height h in model to length or height in prototype is denoted by n, and subscripts p and m of symbols used herein refer to prototype and model respectively, it follows that

$$l_m = n\, l_p \quad \text{(34a)}$$

and

$$h_m = n\, h_p \quad \text{(34b)}$$

TABLE 7.—COMPARISONS OF COMPUTED VALUES BY EQ. 31, HIROI'S

h_d, in cm	B, in cm	1:m	h_1, in cm	T, in sec	H, in cm	H/L	h_1/H
37	10	1:2	13.7	1.5	17.3	0.061	0.79
20	10	1:2	9.0	1.3	6.3	0.028	1.43
20	10	1:3	9.0	1.33	7.6	0.034	1.18
37	10	1:3	9.0	1.5	13.0	0.047	0.69
37	10	1:3	12.0	1.5	15.0	0.052	0.80
37	10	1:3	15.0	1.5	16.2	0.054	0.93
37	0	1:5	9.0	1.21	16.2	0.074	0.56
37	0	1:5	12.0	1.21	17.1	0.081	0.70
37	0	1:5	8.0	1.65	11.6	0.035	0.69
37	0	1:5	12.0	1.65	14.4	0.043	0.83
37	0	1:5	6.0	1.29	14.3	0.064	0.42
37	0	1:5	9.0	1.29	15.6	0.067	0.58
37	0	1:5	12.0	1.29	17.0	0.071	0.71

Denoting time and the intensity of pressure by t and p respectively, the following relationships according as Froude's law of similarity is obtained.

$$t_m = \sqrt{n}\; t_p \qquad (35a)$$

and

$$p_m = n\; p_p \qquad (35b)$$

The water-depth h_m, wave-height H_m, and wave-length L_m used in the experiments were approximately $h_m = 20$ to 70 cm, $H_m = 10$ to 25 cm, and $L_m = 200$ to 450 cm. Those generally used in prototype for the design of breakwaters are approximately $h_p = 5$ to 15 m, $H_p = 2$ to 6 m, and $L_p = 40$ to 90 m. These make the ratio of length in model to length in prototype approximately

$$n = 1/20 \qquad (36)$$

Hence the following values are obtained for the wave-period and the intensity of pressure in prototype.

$$T_p = \frac{1}{\sqrt{n}}\, T_m = 4.47\; T_m, \qquad (37)$$

and

$$p_p = n\; p_m = 20\; p_m \qquad (38)$$

Since

$$T_m = 1.2 \text{ to } 2.0 \text{ sec} \qquad (39a)$$

then

$$T_p = 5.4 \text{ to } 9.0 \text{ sec} \qquad (39b)$$

AND MINIKIN'S FORMULAS WITH EXPERIMENTAL VALUES

Experimental Value $p_{max.}$, in gr per sq cm	$p_{max.}$ by Eq. 32, in gr per sq cm	$p_{max.}$ by Minikin's Formular, in gr per sq cm	p by Hiroi's Formular, in gr per sq cm	Simultaneous Pressure Distribution
169	179	108	26	A, B-type
130	137	34	10	C-type
140	144	41	11	C-type
110	139	52	20	C-type
150	162	80	23	C-type
150	181	107	24	C-type
150	155	82	24	C-type
180	182	124	26	C-type
140	125	34	17	C-type
170	154	66	22	C-type
97	126	45	21	A-type
162	151	74	23	A-type
192	176	109	26	A-type

Since the maximum intensity of pressures observed in the experiments, $\left(p_{max.}\right)_m$, was approximately 200 gr per sq cm, the maximum intensity of pressure in prototype corresponds to

$$\left(p_{max.}\right)_p = 40 \quad \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

in tons per sq m. The values of wave-period and wave-pressure intensity in Eqs. 39 and 40 are considered to include almost the ranges of wave-period and wave-pressure used for the design of breakwaters, though a few prototype observations of $\left(p_{max.}\right)_p$ = 60 to 70 tons per sq m were reported hitherto.

Maximum Pressure Formula for Breakwaters in Prototype.

a. Maximum pressure formula for ordinary breaking waves. The formula for the maximum intensity of shock pressures in tons per sq m exerted by ordinary breaking waves on the vertical walls of composite-type breakwaters in prototype are obtained from Eq. 31 using Froude's modulus of similarity: for the region of $h_1 \frac{h_1}{h_o} \frac{H}{L} \leqq 0.044$ m

$$p_{max.} = 4 + 500\ w_o\ h_1 \frac{h_1}{h_o} \frac{H}{L} \quad \ldots\ldots\ldots\ldots (41)$$

for the region of $h_1 \frac{h_1}{h_o} \frac{H}{L} > 0.44$ m

$$p_{max.} = 20 \text{ to } 26 \quad \ldots\ldots\ldots\ldots\ldots\ldots (42a)$$

as an average value

$$p_{max.} = 23 \quad \ldots\ldots\ldots\ldots\ldots\ldots (42b)$$

Since the value of $h_1 \frac{h_1}{h_o} \frac{H}{L}$ will be greater than 0.044m in most of the conditions used for the design of composite breakwaters encountered with severe storms, the maximum intensity of shock pressures exerted by ordinary breaking waves, $p_{max.}$ may be assumed to be likely equal to 20 to 26 tons per sq m for those conditions without any calculation, and it may be appropriate to use $p_{max.}$ = 26 tons per sq m for breakwaters exposed to an open sea and $p_{max.}$ = 20 tons per sq m for breakwaters located in an inland sea or a bay.

b. Maximum pressure formula for extraordinary breaking waves. For the maximum intensity of much higher shock pressures exerted by extraordinary breaking waves on the vertical walls of composite breakwaters in prototype, the following formulas are obtained from Eqs. 32 and 33 using Froude's modulus of similarity: for average values between the upper and lower limits

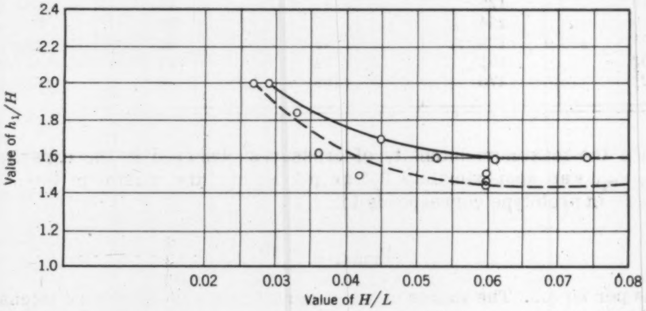


FIG. 15.—RELATIONSHIP BETWEEN H/L AND THE LIMITING VALUE OF h_1/H

$$p_{max.} = 96 \left(0.008 + h_1 \frac{h_1}{h_o} \frac{H}{L} \right)^{1/3} \quad \ldots\ldots\ldots\ldots (43)$$

for the upper limiting values

$$p_{max.} = 103 \left(0.01 + h_1 \frac{h_1}{h_o} \frac{H}{L} \right)^{1/3} \quad \ldots\ldots\ldots\ldots (44)$$

Limit Between a Breaking Wave and a Clapotis.—It has been well known that when the depth in front of a vertical wall increases up to twice or larger than the height of a progressive wave, a standing wave or a clapotis is produced seaward of the wall by superimposing the progressive wave on the reflected wave, producing a weak pressure near the hydrostatic pressure. From a number of the experiments in various shapes of composite breakwaters it was found that when the steepnesses of progressive waves were comparatively large, clapotis were produced in the seaward depths of a vertical wall which were smaller than twice the heights of progressive waves, that is, for such steep waves as $H/L \geqq 0.060$, in depths of $h_1/H \doteq 1.5$ to 1.6, and for waves of such low steepnesses as $H/L \leqq 0.030$, in depths of $h_1/H \geqq 2.0$.

The relationship between wave-steepness H/L and the limiting value of h_1/H in which a clapotis is produced is shown in Fig. 15.

Comparisons of Values Computed by the New Formulas with Observed Pressures at Dieppe.—It is most desirable to check the applicability of the new formulas for the design of breakwaters in prototype by the use of a number of observed data concerning shock pressures exerted by breaking waves during storms, but only very few data are available on the shock pressures of hurricane waves, as far as actual observations are concerned. Only the ten data of actual measurements observed in the harbor of Dieppe by French engineers[7] were used to check the new formulas.

Comparison of the Observed Data in 1937 with the Computed Values.—Pressures observed on the wall of the breakwater in the harbor of Dieppe during the storms of February 9 and 23 and March 1 in 1937 are shown by small circles in Fig. 16 (a).

According to the data reported, the nearly vertical wall of the breakwater was located on a 1:5 slope, the ratio of the length of the slope to the wavelength, $S/L = 0.44$ to 0.50, $h_1/H = 0.61$ to 0.83, $T = 6$ to 6.5 sec (calculated), and $H/L = 0.045$ to 0.063. In consideration of these conditions, the shape of the breakwater and the characteristics of the waves at Dieppe are contained in the range of the experiments which have been performed on a vertical wall located on the 1:5 front slope of the base-mound of a composite breakwater. Therefore, from those experimental results it is predicted that the maximum pressures $p_{max.}$ for most of the cases will be high-intensity shock pressures exerted by extraordinary breaking waves, and the vertical distributions of the shock pressures produced simultaneously with $p_{max.}$ may be likely C-type. With these considerations the maximum pressures $p_{max.}$ and the curves of vertical distribution of the maximum simultaneous pressures can be computed by the use of Eqs. 41 to 44 as well as Eqs. 8 to 13 from the characteristics of waves observed at the same times as the measurements of the pressures, as shown in Fig. 16 (a) by full lines.

As is seen from Figs. 16 (a), the computed values of $p_{max.}$ and P, as well as the predicted curves of the pressure-distributions give comparatively good agreement with the observed values for the case of (1), (2) and (5), but for the cases of (3) and (4), the computed values of $p_{max.}$ and P indicate deviations of minus 54% to 55% from the observed values. For such high-intensity shock pressures as these, Minikin's formula gives too small values to compare.

Comparison of Observed Data in 1935 with the Computed Values.—Since the recording of wave characteristics in December, 1935, were not always carried out at the same times as the measurements of pressures, the pressures and the vertical distributions were calculated by the use of Eqs. 41 to 44 as well as Eqs. 8 to 13 from the characteristics of the two representative waves, one of which was the wave observed at $12^h 15^m$ and was used for the waves on December 2 and the other was the wave observed at $12^h 45^m$ for the waves on December 16. The comparisons of the computed values of $p_{max.}$ and P, as well as the predicted curves with the observed data are shown in Fig. 16 (b). From Fig. 16 (b) it is seen that many of the computed values agree roughly with the observed values. Though the pressure-distribution in the cases of (9) and (10) appears to be A-type at the instants observed, it is supposed from the experimental results such as shown in Fig. 11 (b) that the breaking waves were in a transient state from A-type to C-type distribution.

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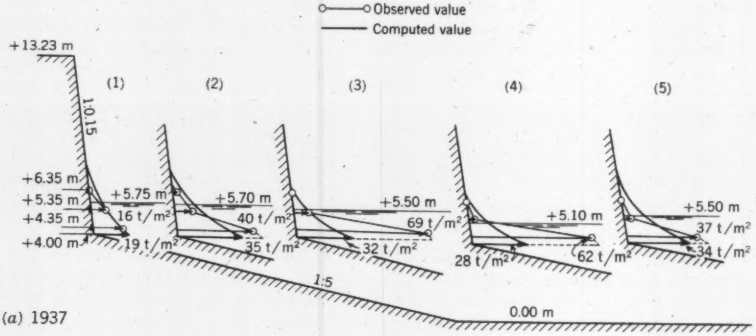
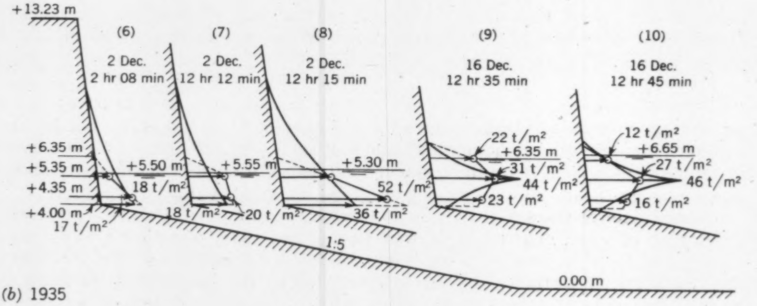
[7] "Etat Actual des Études Internationales sur les Efforts Dus aux Lames," by Rouville, Besson, and Petry, Ponts et Chaussees, July, 1938.

CONCLUSIONS

1. Applicability of the experimental results to prototype. In consideration of the following three facts, it may be correctly considered that the new Eqs. 8 to 13 as well as Eqs. 41 to 44 derived from the experimental data will be applicable for the design of actual breakwaters: (a) since the effect of the frictional resistance on the slope of the base-mound as well as on the bottom and both side-walls of the channel were small, the experimental results may be applicable to prototype conditions with sufficient accuracy by the use of Froude's law of similarity; (b) most of the experimental values of the resultant P of the maximum simultaneous pressures indicated deviations smaller than ±70% from the values computed by Hiroi's and Minikin's formulas derived empirically from the disastrous damages of actual breakwaters; and


Observed value
Computed value
+13.23 m
1:0.15
(1)
(2)
(3)
(4)
(5)
+6.35 m
+5.35 m
+4.35 m
+4.00 m
+5.75 m
16 t/m²
19 t/m²
+5.70 m
40 t/m²
35 t/m²
+5.50 m
69 t/m²
32 t/m²
+5.10 m
28 t/m²
62 t/m²
+5.50 m
37 t/m²
34 t/m²
1:5
0.00 m

(a) 1937


+13.23 m
(6)
2 Dec.
2 hr 08 min
(7)
2 Dec.
12 hr 12 min
(8)
2 Dec.
12 hr 15 min
(9)
16 Dec.
12 hr 35 min
(10)
16 Dec.
12 hr 45 min
+6.35 m
+5.35 m
+4.35 m
+4.00 m
17 t/m²
+5.50 m
18 t/m²
+5.55 m
18 t/m²
20 t/m²
+5.30 m
52 t/m²
36 t/m²
22 t/m²
+6.35 m
31 t/m²
44 t/m²
23 t/m²
12 t/m²
+6.65 m
27 t/m²
46 t/m²
16 t/m²
1:5
0.00 m

(b) 1935

FIG. 16.—COMPARISON OF PREDICTED VALUES WITH OBSERVED PRESSURES

(c) the seven values of P computed by the use of Eqs. 8 to 13 as well as Eqs. 41 to 44 agreed with the corresponding seven observed values of the ten pressure-measurements at Dieppe with plus or minus deviations smaller than 20%.

2. Ordinary and extraordinary breaking waves. Breaking wave striking severely against breakwaters can be divided into the two kinds of ordinary and extraordinary breaking waves.

The ordinary breaking wave includes various types of breaking waves over the wide range between the extraordinary breaking wave and a clapotis, and the maximum shock pressure-intensity $p_{max.}$ for the ordinary breaking wave can be predicted by Eqs. 41 and 42.

The vertical distribution of the maximum simultaneous pressures as well as the maximum resultant pressure P per unit length of breakwaters can be predicted by Eqs. 8 to 13 for both the ordinary and extraordinary breaking waves.

The extraordinary breaking wave is a very severe breaking wave exerting on breakwaters higher shock pressures than twice or more the pressures of the ordinary breaking wave. The very high-intensity shock pressures of the extraordinary breaking wave can be computed by Eqs. 43 and 44.

Though the shock pressures of the extraordinary breaking wave are very large, the probability of occurrence is much smaller than that of the ordinary breaking wave. If the front slope of the base-mound of a breakwater and water depth on the top of the mound (water depth in front of the vertical wall of a breakwater) are designed appropriately, the extraordinary breaking wave will be prohibited from occurring in front of the breakwater.

From this consideration, composite breakwaters located at shallow water and exposed to heavy storms should be designed by the use of Eqs. 41 and 42 for the ordinary breaking waves, by blocking of the occurrence of the extraordinary breaking waves in front of the breakwaters.

3. Shape of a composite breakwater and wave pressure. The inclination to the horizon and the length of the front slope of the base-mound of a composite-type breakwater, as well as the width of the mound-top had a great effect on the intensity and vertical distribution of the shock pressure exerted by a breaking wave. For the base-mounds with the front slopes of 1:2, 1:3, and 1:5 used in the experiments, it was felt that as the slopes became gentle, the intensities of the shock pressures and the probability of occurrence of the extraordinary breaking wave increased. That is to say, there occurred most frequently severe pressures in the cases of base-mounds with a 1:5 front slope.

ACKNOWLEDGMENTS

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ENGINEERING LAW APPLIED TO HIGHWAY DRAINAGE

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SYNOPSIS

Engineering law is defined as the working knowledge of law that the engineer should apply with confidence in his regular practice. Applications to highway drainage are exemplified after introductory definitions of legal terminology. Although some reference is made to California statutes, basic problems depend on widely applicable common law.

INTRODUCTION

In its broadest sense, "engineering law" is the law of the land pertinent to the practice of engineering as it effects an engineer professionally and his works physically, in relation to the public, to the client, to the contractor, their agents or agencies, and their property or other vested rights.

In a more practical sense, the term is limited to that part of the law of the land that the engineer should learn and understand, as an integral part of his professional education and training, with such competence that he can apply it with confidence in his regular practice. He may consult or engage an attorney, as a precaution on important matters, or to interpret new law, or to litigate, but these are infrequent exceptions to his daily exercise of legal judgment on what can or cannot be done lawfully, and how or when to do it.

Law derives from many sources, too many to list in this elementary treatise. For simplicity we will speak of statutory law as that deriving from legislation of constitutions, statutes, ordinances, or codes. Practically as forceful is the common law comprising customs and usages recognized and enforced by courts of law. Statutory and common laws are not always explicit because of contradictions, oversights and ambiguities, so that courts must determine the intent

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of the law makers or the most equitable application to litigants. Such determinations may become accepted as case law, particularly if upheld on appeal.

Drainage law derives primarily from the common law, with more and more equity or case law adapting the common law to complications of modern society. The distinction between the two is not always important in engineering applications, except that common law is a set of broad general principles while case law is a set of specific rules for particular premises. A specific rule cannot be applied to a new case unless the particular premises are essentially the same.

TERMINOLOGY

Most engineers appreciate the importance, particularly in contracts and specifications, of assigning a precise meaning to each generic word. It is equally important in law. In many cases the courts have already agreed on definitions of words or phrases quite different from ordinary engineering usages. For example, the engineer commonly classifies all waters as either surface waters or ground waters, dividing them at the soil or rock surface of the earth, but in law surface waters has a more restricted use. Flood waters, watercourse, erosion and many other words and phrases are similarly used.

Surface waters are those that have been precipitated on the land from the sky or forced to the surface in springs, and which have then spread over the surface of the ground without being collected into a definite body or channel. They appear as puddles, sheet or overland flow, and rills, and continue to be surface waters until they disappear from the surface by infiltration or evaporation, or until, by overland or vagrant flow, they reach well-defined watercourses or standing bodies of water like lakes or seas.

Stream waters are former surface waters that have entered and now flow in a well-defined natural watercourse, together with other waters reaching the stream by direct precipitation or rising from springs in bed or banks of the watercourse. They continue as stream waters as long as they flow in the watercourse, including overflow and multiple channels as well as the ordinary or low-water channel.

Watercourse, in the legal sense, refers to a definite channel with bed and banks within which water flows, either continuously or in season. A watercourse is continuous in the direction of flow and may extend laterally beyond the definite banks to include overflow channels contiguous to the ordinary channel. The term does not include artificial channels such as canals and drains, except as natural channels are lawfully trained or restrained by the works of man. Neither does it include all depressions or swales through which surface or errant waters pass.

Flood waters are former stream waters that have escaped from a watercourse (and its overflow channel) and flow or stand over adjoining lands, and they remain as such until they disappear from the surface by infiltration, evaporation, or return to a natural watercourse. They do not become surface waters by mingling with such waters, nor stream waters by eroding a temporary channel.

Navigable waters are those stream waters lawfully declared or actually used as such. Navigable waters of the State of California are those declared to be such by statute. Navigable waters of the United States are those determined by the Chief of Engineers to be so used in interstate or international commerce. Other streams have been held navigable by courts under the common law that navigability in fact is navigability in law.

Diversion has two distinct meanings, which will be clear in context. In one, it is the taking of water from a stream for a beneficial purpose (irrigation, water supply, power, and so on), even though a portion may return to the same stream. The other is the deflection of surface waters or stream waters so that they discharge into a watercourse to which they are not naturally tributary. Deflection of flood water is not diversion.

Erosion and scour are the cutting or wearing away by the force of water of the banks and bed of a channel in horizontal and vertical directions, respectively.

Erosion and accretion are loss and gain of land, respectively, by the gradual action of a stream in shifting its channel by cutting one bank while it builds on the opposite bank. Property is lost by erosion and gained by accretion, but not be avulsion when the shift from one channel to another is sudden. Property is gained by reliction when a lake recedes.

Backwater is an unnaturally high stage in a stream caused by obstruction or confinement of flow, as by a dam, a bridge, or a levee. Its measure is the excess of unnatural over natural stage; it is not the difference in stage upstream and downstream from its cause.

Concentration, in addition to its general sense, means the unnatural collection or convergence of waters so as to discharge in a narrower width, and at greater depth or velocity.

Easement is a right to use the land of others. It may derive from the common law or be acquired. It is usually acquired by purchase or condemnation, but occasionally by prescription or inverse condemnation. The right is not exclusive, but subject to rights of others in the same land, the lesser right being servient to a prior right which is dominant. Easements for drainage may give rights to impound, divert, discharge, concentrate, extend pipelines, deposit silt, erode, scour, or any other necessary consequence of a highway development.

Use of land of others without right usually leads to right in the future. If use is adverse and notorious for a statutory period, an easement is acquired by prescription, without compensation, but at any earlier time the owner of the other land may sue for compensation by inverse condemnation.

Ground water is that situated below the surface of the land, irrespective of its source and transient status.

Subterranean streams are flows of ground waters parallel to and adjoining stream waters, and usually determined to be integral parts of the visible streams.

Percolating waters are those that have infiltrated the surface of the land and moved slowly downward and outward through devious channels (aquifers) unrelated to stream waters, until they reach an underground lake or regain and spring from the land surface at a lower point.

Artesian waters are percolating waters confined below impermeable formations with sufficient pressure as to spring or well up to the surface.

Perched waters are percolating waters detained or retained above an impermeable formation, so as to stand above and detached from the main body of ground water.

Swamps are lands saturated by ground water standing at or near the surface. (If the water has risen to the surface at a fault, the swamp is a cienega.)

Marshes are lands saturated by waters flowing over the surface in excess of infiltration capacity, as in sloughs of rivers and tidal channels.

Owner herein means any owner of land, usually specified in relation to another owner. Of two owners affected by flow of water, the one upland is the upper owner, the other the lower owner. The highway has an owner, with the same rights in common law as private owners.

LAW AND FACT

The engineer applying rules of law to highway drainage must bear in mind the distinction between law and fact. The matter is most important in litigation, in which questions of law are decided by the judge and questions of fact are resolved by the jury. The judge is an expert on questions of law and he can hardly overlook any pertinent statute or precedent when two aggressive and highly competitive attorneys lay their briefs before him. On the other hand, the jury is likely to have little qualification, individually or collectively, to decide questions of fact. There will be contradictory and confusing testimony on each question with qualified experts contradicting each other, and the attorney with the weakest evidence trying deliberately to confuse the issue.

One such question of fact is whether or not certain water was flowing in a natural watercourse; if so, the waters were stream waters and damage was compensable; if not, the waters were flood waters and damage was not compensable. A 7-to-5 vote one way or the other might determine the entire case. Backwater is also a question of fact. With few exceptions, highway crossings restrict or obstruct channels and cause some backwater. If the highway engineer testifies that the backwater is 2 in. and plaintiff's engineer calls it 2 ft, the jury has to decide which expert is right.

The most important question of fact decided by the jury is the amount of the damage. It is notorious that if a private owner sues the state and wins, the award is generous, but if the state is the injured party with the same apparent facts, the state loses or collects only a nominal award. So the engineer must not only distinguish between law and fact, but must also be aware of the "facts of life."

SURFACE WATERS

By the common law, the upper owner has an easement over lands of a owner for surface waters to flow or escape from his land by natural ways and routes.

The upper owner may not, without liability, change the point of discharge of surface waters, nor concentrate them in ditches, nor divert in that direction waters that would have escaped in another direction, nor discharge them at higher velocity, nor add to their pollution. He may, however, increase their quantity by roofing or paving over pervious soils, or by levelling his land so as to eliminate puddles and ponds. Likewise, he may decrease the quantity by retention for his own use, but he will then risk loss of his easement.

The lower owner may not, without liability, obstruct natural flow of surface waters on to his land, either by excluding it or causing backwater on his neighbor.

The highway owner may be in either position, with substantially the same rights and liabilities in law. As a lower owner, he intercepts and receives surface waters in cuts and must arrange to pass such waters under his fills. As an upper owner, he must add to the waters so received the on-site precipitation, and discharge all to a lower owner as he would have received them naturally. He may discharge them in some other way without damage, or in some damaging way with compensation. The fact that pavement is impervious and sheds more of the precipitation does not add to liability, per se.

This problem of disposal of surface waters is one of the most difficult tasks in highway design. If the lower owner formerly received them as sheet flow, it will generally be impractical to spread the water intercepted by and precipitated on the highway and collected in gutters, ditches and other drainage facilities in the same way. In a thorocut, water would have to be pumped over the lower face of the cut. On a sidehill cut, the water would have to be discharged over the outer shoulder to run in sheet flow down the fill slope. On a thorofill, water carried through the fill in conduit would have to be dispersed in some sort of an outlet trough to spread out on the lower land.

The usual solution is to collect the surface waters in small concentrations and convey the concentrations to a natural watercourse. In built-up areas, the waters may be discharged in a storm sewer. In rural areas there may be no objection to discharge of small concentrations on marginal lands. Use is also made of small spreading areas and recharge wells to sink the water into the ground. In mountain and desert areas the problem is obviously simpler. Unusual care must be taken in suburban areas where future changes in land use must be anticipated.

The foregoing applies to state and county highways, but procedures are usually different in municipalities. If by ordinance a city establishes grade along the property line of a street, it can improve the street to that grade without liability to owners of abutting property lying above or below the established grade. Also the pattern of a street system is more adaptable to disposal of collected surface waters than the pattern of rural highways.

An analogous procedure is used in subdivisions, even outside of incorporated cities. By grading all streets before selling lots, the subdivider effectively establishes a de facto grade that purchasers of lots accept, as well as the consequent drainage.

STREAM WATERS

Where natural watercourses are unquestioned in fact and in permanence and stability, there is little difficulty in applications of law. Highways cross the channels on bridges or culverts, usually with some constriction of the width of the channel and obstruction by substructure within the channel, both causing backwater upstream and acceleration of flow downstream. These changes in regimen must be so small as to be tolerable by adjoining owners, or these owners must be compensated.

Highways often discharge surface waters into the most convenient watercourse. The right is unquestioned if those waters were naturally tributary to watercourse and unchallenged if the watercourse has adequate capacity. However, if all or part of the surface waters have been diverted from another watershed to a small watercourse, any lower owner may complain and recover for ensuing damage.

Applications of law become more complicated when the regimen of a channel is changed by other owners. If, for example, upper owners change the character of the watershed so that stream waters are increased in volume, lower owners are obliged by the common law to accept the increase, there being no diversion. They are not obliged to improve the channel through their lands, although they may choose to do so for self protection. Unsettled is the question of whether or not an upper owner can compel a lower owner to improve his formerly adequate channel if the increased flow is detained and backs up on lands of the upper owner. Where the lower owner is the highway which had provided an adequate bridge or culvert, some contend that the facility should be enlarged promptly on demand of the upper owner, while others insist the highway has the optional alternative of maintaining the overtaxed facility, at least until some substantial change was required by reconstruction of the highway itself, or retirement of the facility at the end of its useful life. The latter contention is supported, for state highways, by constitutional law against use of highway funds for nonhighway purposes, that is, for benefit of private property.

Equally important and also unsettled is the right of a lower owner to completely obliterate a natural watercourse in the process of planing his land for agricultural purposes. Section 725 (b) of the Streets and Highways Code of the state of California is clear on the law, but the question of there having been a watercourse is one of fact left to a jury. (The citations referred to herein all relate to the laws of the state of California.)

Law does not define any measure of adequacy for improved channels. The highway does not escape liability for obstruction of a channel by providing a bridge or culvert to convey a 30-yr flood, or a 100-yr flood, or even a 1,000-yr flood. In common law, such an extreme event might be called an "act of God," defined as "a direct, sudden, and irresistible action of natural forces, such as could not humanly have been foreseen or prevented." However, such a plea is a weak defense in claims for damages, for engineers foresee and can provide for stream waters of great magnitude. Law does not prescribe the provision, which is left to engineering economics.

FLOOD WATERS

The law with respect to flood waters is the simplest of all, but the definition of such waters must be read with attention to every word. They are not simply storm waters, or floods, cloudburst floods, or overflows, errant, migrant, or vagrant waters. Surface waters do not become flood waters, no matter how fast, or deep, or where they flow, unless enroute they have entered a natural watercourse and escaped. They have not escaped if they run in an overflow channel or in an outer slough of a threaded channel. They are flood waters only if they have been stream waters and have completely escaped from the natural watercourse, including its collateral channels.

In common law, such waters are a "common enemy" of all people, lands, and property attacked or threatened by them. Everyone, including owners of highways, can act in any reasonable way to protect himself and his property from the common enemy. He may obstruct its flow from entering his land, backing or diverting water onto lands of another without liability. He may discharge such water from his land onto the land of his neighbor, without penalty, by gravity or pumping, by diverting dikes or ditches, or any other reasonable means. "Reasonable" appears to mean any expedient except destroying your neighbor's defenses, as by dynamiting his levees.

By the definition, flood waters are not necessarily infrequent occurrences. Where frequent, the highway may be designed to obstruct, divert, or pass the flow, without liability to abutting or other owners. There is some risk that frequent flow of flood waters might lead to finding by a jury that the path was in fact a natural watercourse. Such paths have become main channels (such as the Los Angeles River and Los Gatos Creek).

GROUND WATERS

Any interference by a highway with flow or stage of ground waters may be cause for complaint with claim for relief. Excavations, particularly thorocuts, have drained away ground waters that had been pumped for irrigation supply. The supply might remain adequate at a lower level, increasing pump lift.

Embankments may compress underlying water-bearing soils and restrict the percolation of ground water from one side to the other. On the side of the source, the water table may rise to the surface and waterlog the land, while on the other side wells may go dry and owners may be compelled to drill deeper and pump higher.

Both state and local laws restrict operations which may pollute or contaminate ground waters used for domestic or irrigation purposes. Plans for disposal of surface waters by spreading ponds or recharge wells must comply. There are allegations of three hazards: (1) contamination from highway litter may reach and spread in the ground water, (2) hydrocarbons from road oils and vehicle exhaust may mulsify in and clog aquifers, and (3) drainage from road metal may clog aquifers by calcination.

UNNATURAL WATERS

As a general rule waters other than flood waters which have been forced out of natural pools or paths by the works of man become a responsibility of some person or agency who must divert, store, use, and waste such waters without damage to others. Highways may be affected in all degrees from the monumental reservoir which may inundate a roadway down to the sprinkling of lawns so as to drain excess water on the pavement. Intermediate are canals carrying water for irrigation of land, drains carrying the return water back to natural channels, and conveyance and ponding of water for power, mining, navigation, flood control, industry, sanitation and recreation. The common law has been clarified and modified by statute, recognizing some uses as higher than others.

The Streets and Highways Code of California, §725-730 and §1487-1488, protect state and county roads, respectively. No one may drain or permit drainage of (unnatural) water on highways (725a), nor store or distribute (unnatural) water so that it overflows or seeps onto the highway (725c), if damage results, and the State may act to obtain relief (726-727). Negligent, willfull, or malicious acts are misdemeanors (1487 and Penal Code §588). Note, however, that city streets are not covered by these statutes; unnatural water is usually allowed to flow into gutters and storm sewers, but such flow may be restricted by local ordinance.

Enforcement of these statutes is not simple, even for flagrant violations. Most abutting owners, through misinformation, believe highways, like city streets, are public drains. Jurors have similar beliefs: even after stern instruction by the Court, they remain sympathetic and return verdicts for only nominal damages. Application of these laws by the maintenance engineer can be greatly simplified if the design engineer establishes grade higher then abutting lands and provides adverse grade and roadside dikes to exclude drainage from depressed roadways.

Several statutes govern canal and ditch crossings in California (S&H Code §728-729, 1489-1490; Water Code §7030-7033). The general rule may be stated, "First in right is first in might." That is, if the canal is there first, the highway must cross it without undue obstruction of flow, but if the highway is there first, the canal owner must provide a crossing without undue interference with traffic. In each case the new tenant pays the cost and is responsible for future maintenance, repair, replacement and removal if abandoned. An exception is specified in WC §7033 for permanent construction.

These statutes are not specific for expansion of facilities: that is (1) if a canal is built through an existing highway and the highway is widened, who pays for widening the canal bridge; or (2) if a highway crosses an existing canal and the canal is enlarged thereafter, who pays for the enlargement of the bridge? An obvious answer is that the active party always pays unless the passive party is specifically obligated. For example, if a highway has secrued right of way for a divided roadway but built only half when a canal builds across, the permit for the canal may require immediate or future construction of a facility for the ultimate highway construction.

STATUS OF CHANNELS AFTER SUSTAINED CHANGE

Channels may lose their character as natural watercourses through natural or artificial changes that appear to be permanent. Natural avulsions of short reaches of meandering streams are quite common. Complete diversion of a stream (Owens River) was held to quit former rights to discharge water in its former channel or terminal lake. It has been contended that partial regulation of a stream so as to limit its maximum discharge abandons the right to restore the full natural flow at a later date, but this question is unsettled.

Application of this rule and the latter contention to highway practice may be confusing. Obliteration of a watercourse by land planing on one side may impound water against the other side of the highway and over abutting lands, damaging those lands and diverting overflow to another watercourse. The highway owner may be held at fault for not acting to stop the land planing. As an example of the other matter, when upper owners complained that a highway culvert was too small, causing overflow of their lands, plans for enlarging the culvert were stopped by threat of injunction by a city (Fairfield, Calif.), which alleged that lower owners in the city would be injured by overtaxing of the channel that had been improved to match the capacity of the culvert.

These contingencies cannot be entirely avoided, but their number should be substantially reduced by alertness and prompt action by maintenance engineers. Land planing is tolerated in its effect on surface waters, but S&H

§725 (b)(1) forbids it for natural watercourses unless other drainage is provided.

As a corollary, artificial substitutes for natural watercourses may be held to be natural watercourses after long sustained use. There is no statute or other rule to fix the minimum of time to measure permanence of change, but it may be presumed that a court would consider prescriptive periods, or general acceptance of the change, or obliteration of other channels.

INVERSE CONDEMNATION

In most matters, public agencies cannot be sued without their consent. An important exception is the suit in inverse condemnation, alleging that private property has been taken without just compensation. The agency could have taken the property by eminent domain, by a suit in condemnation to prove public necessity for the taking, and then determine the value of the property taken. However, often, some properties are overlooked or considered speculative or intangible, so that the owners initiate the eminant domain action inversely. Easements for drainage are notable among such properties.

Engineers will frequently be called to prepare and present evidence in the defense of such suits, at which time they will be well advised by attorneys. The real defense, however, should begin long before suit is filed, at a time when the engineer is acting on his own initiative. This defense is an accurate knowledge of the conditions of lands along the projected highway, so as to show the change in conditions after the highway has been constructed. For lack of good evidence (such as maps, photos, notes of competent observers) juries will believe the highway caused all damage cited in the complaint.

LIABILITY FOR NEGLIGENCE

Damage to private property by flooding or erosion may be due to negligence instead of an involuntary taking of a drainage easement. Complaints of damage due to negligence must allege fault or default of one or more employees, contending the following as a minimum (Government Code of California §1953): (a) there was a defect and it caused damage, (b) an employee was responsible for the defect, or had notice of it, (c) he had authority, duty and funds to repair it, (d) he did not promptly repair or warn of the defect, and (e) claimant was not also negligent.

Negligence in highway drainage may derive from errors of omission or commission in design, construction, operation or maintenance. In none of these fields is the engineer infallible in skill or judgment, but he must have acted with the care, skill, judgment, and diligence ordinarily exercised by professional engineers. Deficiencies that have caused complaint are: (a) Poor estimate of design discharge, (b) not conveying discharge to an acceptable outfall, (c) allowing inlet to become obstructed, (d) allowing pond to form where a person drowned, (e) building a weak structure, (f) building an open storm inlet too near the highway, (g) not providing drainage for a depressed street,

(h) underestimating size and volume of drift, or (i) permitting traffic through dangerous overflow.

CALCULATED RISKS

Although more closely related to engineering economics, the taking of calculated risks must be considered an important item in drainage law. The question may arise as to liability for negligence.

The expression is confusing to some engineers because of the use of the word calculated in a sense unusual to them. It does not mean computed on a calculating machine, nor even reduced to numerical values, although such techniques may be part of the calculation. It really means considering with knowledge, prudence and judgment all the hazards, perils and other risks involved in a proposed course of action, together with difficulty of avoiding these risks. Taking the calculated risk means acceptance of the risk instead of a different course of action with less risk but greater difficulty.

This is permissible in law. In drainage, particularly, there is no course of action without risk, even if we design for the maximum possible flood. Economy demands design for a smaller flood: prudence demands design for a flood which does not recur too frequently. Between those two indefinite extremes, there must be a sound basis for design. If all the hazards and perils could be foreseen precisely and expressed numerically, the design could mathematically minimize cost of facility plus present value of the risk. Unfortunately these risks are highly speculative, and the engineer is forced to a calculated risk.

The question of negligence arises after an injury allegedly caused by a deficiency in design. Not to have foreseen the injury might have been negligence, but to have foreseen the possibility and weighed it with other factors is a proper exercise of judgment. If the injury is to real property, there may have been a taking, leading to compensation by inverse condemnation, but injury to a person or a vehicle depends on proof of negligence.

REGULATION BY PUBLIC AGENCIES

In addition to proprietary rights, which are a matter of record, many public agencies have responsibility and authority over certain operations on natural waters and channels that may affect highway drainage. Without exaggeration, the number of such agencies in California exceeds a thousand. Fortunately, the complexity due to numbers of agencies is offset by specific definition of their powers in statutes.

One agency is the State of California Division of Lands, which administers the state's ownership of beds of navigable waters of the State of California, listed in the state's Harbor and Navigation Code §102-106. Its permit or easement must be obtained for crossings over such waters, channel changes, dredging, and so on, (S&H Code §101.5).

Another is the State of California Reclamation Board with its regulatory powers (Water Code §8710) over operations "in the bed of or along the banks of the Sacramento or San Joaquin Rivers or any of their tributaries or connected therewith, or upon any land adjacent thereto, or within any of the overflow basins therein, or upon any land susceptible to overflow therefrom." This broad definition covers all of the central valley of California, from Shasta to

Tehachapi and Clear Lake to Donner Summit, within which, literally, permission must be sought for every embankment or excavation, even to digging a hole to plant an oleander in a highway median. Obviously, administration is not so literal, being limited to the intent of the law to improve and maintain natural channels, canals, drains, and floodways by dredging and leveeing. Pending draft of a more realistic definition, an informal procedure has been set up at staff level for advance determination of the Board's interest in particular channels. Cooperation with highway funds is limited by the state of California's S&H Code §101.3.

In highway drainage, water is a liability, but drainage works cannot ignore the value of water as a natural resource. The California Department of Water Resources administers the right to use water and must be consulted when drainage plans conflict with conservation or distribution projects. The department is also responsible for supervision of dams, and although highway embankments are specifically excluded from the definition of a dam (California Water Code §6004), check dams require permits if certain minima are exceeded (a 6-ft dam impounding 50 acre-ft or a 25-ft dam impounding 15 acre-ft, (California Water Code §6002-6003).

The California Department of Fish and Game is interested in natural waters traversed by anadromous fish. Migration of fish may be stopped at highway culverts, particularly at low water, if the invert is flat so as to shoal the flow, or if excessive drops cannot be jumped by the fish. There is no statute requiring adaptation of culverts to migration of fish, so highway funds cannot be used for the purpose. However, it will usually be possible to establish flow line and shape invert to accommodate fish at no extra cost. Inverts have been built V-shaped. Multiple culverts have been built with one unit lower than the others. Flow lines have been lowered to avoid drop to a pot-hole at the outfall. Usually unacceptable are proposals for fish ladders and slalom battens inside the culvert. Not only do these require improper use of highway funds, but they trap drift and detritus, reducing the capacity of the culvert and adding to the cost of maintenance.

Mention should also be made of the California Water Pollution Control Board, and its regional agencies. No specific statutes or regulations require highway cooperation, but drainage facilities in certain locations may violate the general provisions of the law (the California Water Code §13000 et seq). Flushing hydrocarbon residues from highway pavements into natural watercourses may be "pollution." Detention at highway culverts of waters polluted by others may be "nuisance." The law is too new to predict interpretation by the courts.

Most numerous of the agencies referred to are the "districts" organized under state law. Geographically, they are widely distributed over the state, vary in size from less than 1 mile to more than 100 miles long, and frequently overlap for two or more distinct purposes benefitting the same lands. The following classes of districts may have a regulatory or discretionary interest in highway drainage:

County Flood Control Districts	Water Code §8110
Special Drainage District	8500
Irrigation Districts (112)	20500
County Water Districts (90)	30000
California Water Districts (21)	34000
California Water Storage Districts (4)	39000

Reclamation Districts (180)	50000
County Waterworks Districts (63)	55000
County Drainage Districts	56000
Drainage Districts (20)	Deering 2200-03
Flood Control and Flood Water Conservation (3)	9178
Levee Districts (11)	4284
Municipal Water Districts (17)	5243,9131
Protection Districts (2)	6172,74,75
Storm Drain Maintenance Districts (14)	2208
Storm Water Districts (8)	6176
Water Conservation Districts (16)	9127
(S) Conservation District (1)	4025
(S) County Flood Control District (6)	3515 et al
(S) County Flood Control and Water Conservation (14)	205 et al
(S) County Storm Drainage District (1)	1657
(S) County Water Agency (2)	6730,7303
(S) County Water District (2)	5683,9124
(S) Flood Control District (2)	320,6749
(S) Sanitation and Flood Control District (1)	8934

in which (S) indicates districts created by special statute, and the number of districts active in 1953 are shown in parentheses.

Many federal agencies are concerned with drainage affecting proprietary rights in public lands and reservations, but only one need be mentioned for its jurisdiction over other matters affecting drainage. The United States Army Corps of Engineers is responsible for administration of navigable waters of the United States, for operation of federal flood-control projects, and for control of debris in certain streams.

The Corps is currently classifying navigable waters into two groups. One will be those used only by small craft, such that highway structures clearing flood stages will assuredly provide adequate clearance for navigation at ordinary stages; no formal permit will be required. For the other group, the procedure will be formal, beginning with a detailed application for permit to cross, public hearing to weigh conflicting interests of highways and navigation, economic analysis of relative costs and benefits, and ending with a determination of minimum horizontal and vertical clearances.

On federal flood-control projects, the Corps builds dams and improves channels for operation by local agencies according to a specified set of regulations. The local agency becomes the proprietor with authority to approve plans for highway crossings of flood-control channels, but it will usually refer applications to the Corps for comment before taking action. Preliminary informal examination of alternatives directly with the Corps will expedite subsequent formal action.

With respect to all public agencies, their works and most of their projects are matters of public record. Courts have held that highway engineers should recognize the probable effect of completed works and assured projects in their own design of drainage facilities. It is a matter of discretion, however, as to whether a project is assured at any particular stage of investigation, authorization, appropriation, or construction. Several large dam projects were abandoned after attempted construction disclosed weak foundation rock: debris reservoirs have filled up in the first storm: completed levees have been overtopped.

CONCLUSIONS

In this cursory treatment of drainage law, the principles that can be applied by the highway engineer in his everyday practice have been emphasized. If engineers hesitate to apply these principles without counsel, they should recall the classical proof that even lawyers are right only half the time, because every case has both a winner and a loser. Also, the engineer seeking counsel on every legal question may often be advised not to do something because of uncertainty of right. Unless he knows the degree of uncertainty, he cannot take the calculated risk which may be the most economic solution.

No attempt has been made to cite cases in support of principles of law. Other sources[2,3,4] have done so and should be consulted for further understanding of the exact circumstances which prompted statements of principle.

References to statutes are limited to laws of the state of California. Those from the Streets and Highways Code are similar in many other states, but those from the Water Code are similar only in other western or arid states. In any case, applications of principles should conform to local law.

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[2] "The Engineer at Law," by C. B. and J. R. McCullough, Vol. 1, Oregon, 1945.

[3] "Memorandum Discussion of Rules of Law Respecting Waters, Applicable to Problems of Highway Drainage," by Frank B. Durkee, Calif. Div. of Highways, May 29, 1946.

[4] "The Legal Aspects of Highway Drainage Engineering," by Spencer M. Williams. 9th Annual Street and Highway Conf., ITTE, Berkeley, Calif., January 24, 1947.

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TRANSACTIONS

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NATURAL BY-PASSING OF SAND AT COASTAL INLETS

By Per Bruun,[1] F. ASCE and F. Gerritsen[2]

With Discussion by Messrs. J. R. Bowman; and Per Bruun and F. Gerritsen.

SYNOPSIS

At coastal inlets two main principles in by-passing of littoral-drift sand by natural action occur. These are by-passing on offshore bar and by-passing by tidal flow action. This paper examines these two means of by-passing as a function of littoral drift and tidal flow characteristics of the inlets studied.

INTRODUCTION

By-passing, as used herein, refers to the manner in which the material, after a short interruption caused by an inlet, pass, channel, jetty or other kind of "littoral barrier," is returned to the normal littoral-drift zone a short distance downdrift from the littoral barrier.

If nature did not by-pass sand across inlets, passes, and channels on sea shores a number of marine forelands including barriers, spits, and entire peninsulas would not exist. A typical example of this is found in Florida which was built up of sand washed down by rivers and streams from the Appalachian Highland and carried southward, crossing estuaries and tidal inlets, for final deposition in the huge barrier and ridge systems that is Florida.

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The two main principles in by-passing of sand by natural action are: (1) By-passing on an offshore bar, and (2) by-passing by tidal flow action. Most cases present a combination of these methods.

A submerged bar in front of an inlet or harbor entrance on a littoral-drift coast will often function as a bridge on which sand material is carried across the inlet or entrance. Every channel dredged through the bar will, therefore, be subject to deposits.

By-passing by tidal-flow action takes place when littoral deposits caused by flood currents are spoiled out of the inlet again by ebb currents in the downdrift direction. Both bar and tidal flow by-passing include cases with irregular transfer of large amounts of materials in migrating sand humps or by change in the location of channels.

One can distinguish between inlets or entrances with predominant bar by-passing and inlets that may present a predominant tidal-flow by-passing by considering the ratio $\frac{M_{mean}}{Q_{max}} = r$ between the magnitude of littoral drift (M_{mean} in cubic yards per year) and the quantity of flow through the inlet (Q_{max} in cubic yards per second under spring-tide conditions).

If this ratio is high, bar by-passing is predominant. A low ratio indicates that conditions favoring predominant tidal-flow by-passing exist. Meanwhile whether or not such by-passing actually takes place depends on whether or not it is possible to use the tidal flow for transferring material in the downdrift direction. This depends, among other things, on the inlet configuration. Inlets exist which, due to strong tidal currents, are detrimental to any transfer of material because ebb-currents jet the material so far out into the ocean that it is lost forever to the shore.

Table 1, which will be subsequently presented, gives values of M_{mean}, Q_{max}, and r for various inlets including those to be described. It is shown that inlets with $r < 10 - 20$ have a predominant tidal flow by-passing while inlets with $r > 200 - 300$ have predominant bar by-passing.

BAR BY-PASSING: LIMITED TIDAL ACTION

The Principal Involved.—Fig. 1 shows a barrier with an inlet. Littoral drift material passes along the barrier. At the downdrift end it continues on its way across the inlet on a submerged bar, the extent and depth of which depends on the amount and character of the material that by-passes and the intensity of wave and current action. By increasing the amounts of littoral materials the bar area increases and depths decrease. Increase in wave action results in a smaller and more streamlined bar with greater depth. Stronger longshore currents also result in a narrower and more streamlined bar with perhaps one predominant channel, while weaker longshore currents may result in more and shallower channels. The mechanics of bar by-passing may be a more or less continuous process or it may partly take place in greater irregular sand waves or humps that migrate across the inlet on the bar.

Thus, it is clear that if satisfactory natural by-passing is to be established the criterion must be that the longshore drift capacity be kept on the same level, regardless of the existence of the inlet. Knowing the distribution of littoral drift and longshore currents at different depths in the normal beach and bottom profile, it is possible to estimate the necessary depth for a certain

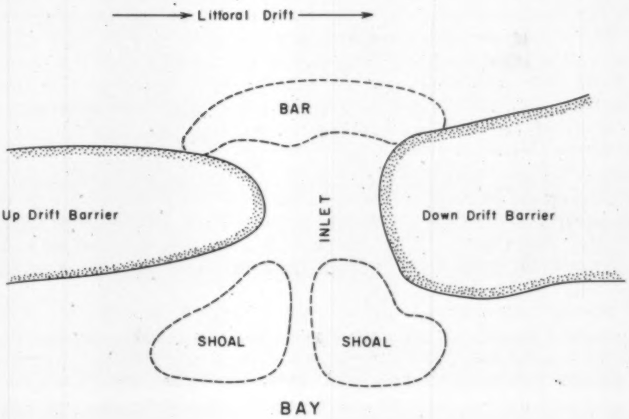
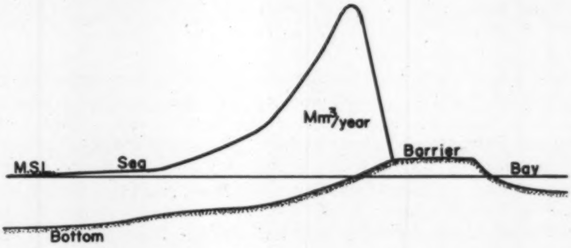
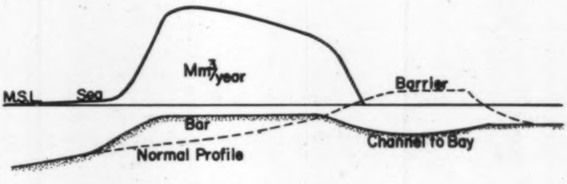

Littoral Drift
BAR
Up Drift Barrier
INLET
Down Drift Barrier
SHOAL
SHOAL
BAY

FIG. 1.—INLETS WITH STRONG AND INLETS WITH WEAK LONGSHORE LITTORAL CURRENT


Mm³/year
M.S.L.
Sea
Barrier
Bay
Bottom

(*a*) NORMAL PROFILE


Mm³/year
M.S.L.
Sea
Barrier
Bar
Channel to Bay
Normal Profile

(*b*) PROFILE AT ENTRANCE BAR

FIG. 2.—THE PRINCIPLE INVOLVED IN BY-PASSING OF SAND ON A SUBMERGED BAR

width or the necessary width for a certain wave action and longshore current action (Fig. 2). Meanwhile, wave-and current-action vary and this will cause irregularities in the by-passing. Experience demonstrates that bar by-passing cannot exist without considerable wave-action and, moreover, that the depth over the bar is usually limited to the breaker depth for predominant storm waves. Width of the bar is, as previously mentioned, mainly a function of the velocity of the longshore current, which in turn is a function of wave characteristics.

It is interesting to compare the mechanics of bar by-passing with the natural by-passing around the California promontories as described by Parker Trask.[3] He notes that the active by-passing zone seems to extend to a 30-ft depth, and even up to about 60 ft depth transferred by the waves stirring up material to be moved by currents.

It is obvious that man has few chances to arrange satisfactory navigation and by-passing conditions simultaneously, because breaking waves that are necessary for by-passing are hazardous to navigation. This is why inlets with bar by-passing are usually only useful for small crafts. Navigation improvements have only limited interest and often are difficult to justify and finance.

Examples Of Inlets With Bar By-Passing.

Unimproved Conditions.—Figs. 3 and 4 show the Matanzas and Ponce de Leon Inlets, respectively, on the Florida east coast where littoral drift is approximately 400,000 cu yd to 500,000 cu yd per yr to the south. Both of these inlets are old and have probably been almost stationary at their present location for hundreds of years. The normal tidal range is approximately 3 ft, but tidal prisms are rather small in both cases.

Matanzas Inlet with 3 ft to 5 ft depth on the bar is an example of irregular bar by-passing. A shifting channel sometimes gets too close to the downdrift beach thereby causing serious erosion of this beach (1955 - 1958). The inlet is not useful for navigation because of its ocean-side shoals.

Ponce de Leon Inlet has a half-moon-shaped bar with a depth of 8 ft to 10 ft. This bar is unstable because of shifting channels. The transfer of sand is irregular causing irregular erosion of New Smyrna Beach on the downdrift side. During the war attempts were made to maintain a channel across the shoals but efforts were not successful.

Oregon Inlet in North Carolina is depicted on English maps from the 16th century. The tidal range is approximately 5 ft and wave action is heavy with 10 ft to 15 ft storm waves. Fig. 5 shows aerial photographs taken in 1945 and 1949. During this period the inlet migrated southward several hundred feet by extension of the northern barrier and erosion of the southern. At the same time the channel across the outer half-moon-shaped bar, with an 8 ft to 10 ft depth, shifted northward, as a result of a great amount of sand was transferred across the inlet. Sand in this case is by-passed in three different ways: (1) In suspension directly across the inlet, (2) by shifting of channels, and (3) indirectly by erosion of the tip of the downdrift barrier which benefits the downdrift beaches. Since 1846 the inlet has migrated 1.5 to 2 miles southward, but in the 1952 to 1958 period 2,500 ft was cut off the northern tip, widening the inlet considerably.

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[3] "Movement of Sand Around Southern California Promonotories," by Parker D. Trask, Beach Erosion Bd., Tech. Memorandum No. 76, 1955.

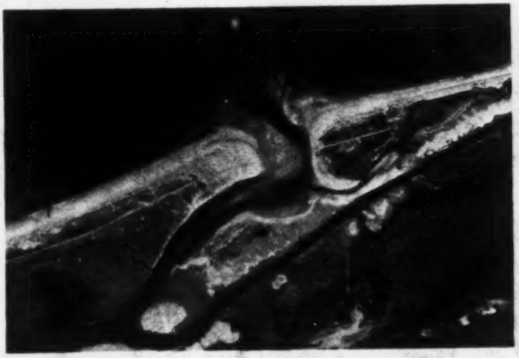

FIG. 3.—MANTANZAS INLET, FLORIDA

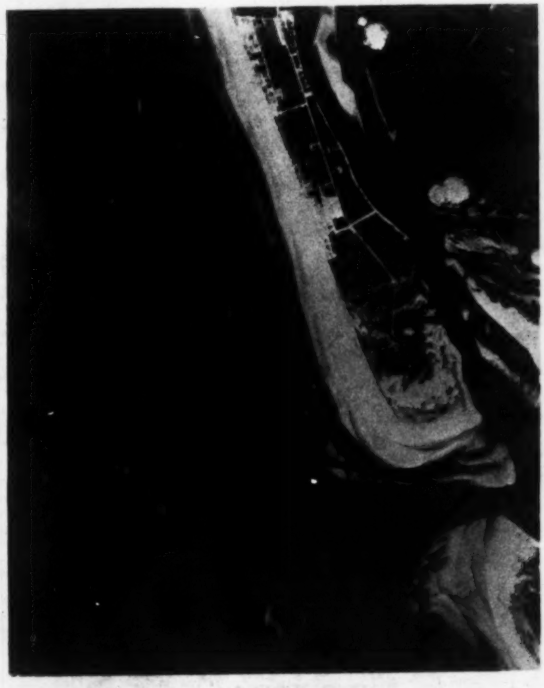

FIG. 4.—PONCE DE LEON INLET, FLORIDA

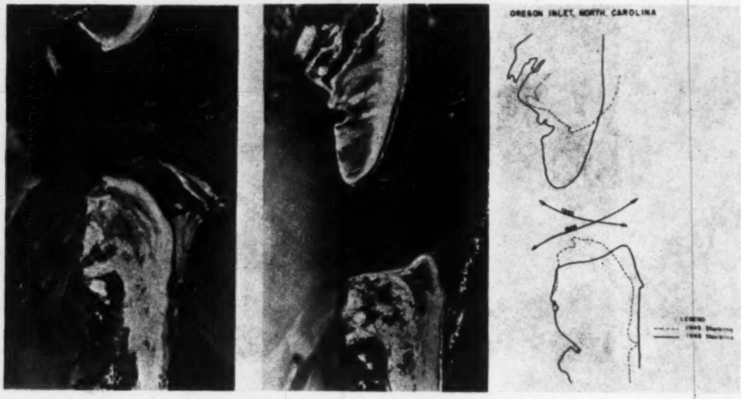
OREGON INLET, NORTH CAROLINA
LEGEND

FIG. 5.—OREGON INLET, NORTH CAROLINA

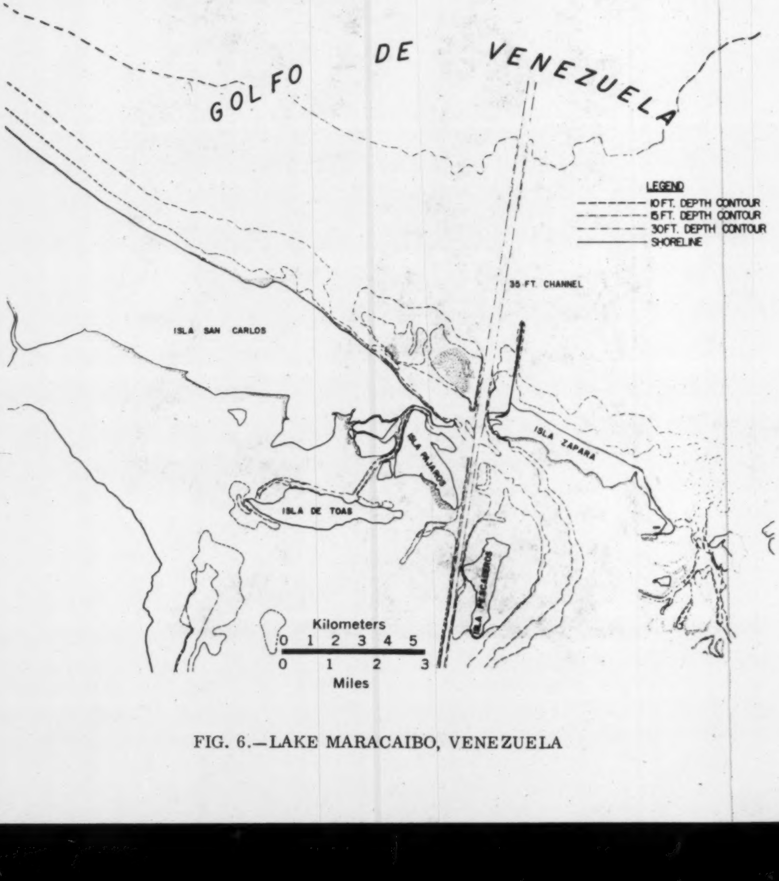
GOLFO DE VENEZUELA
LEGEND
10 FT. DEPTH CONTOUR
15 FT. DEPTH CONTOUR
30 FT. DEPTH CONTOUR
SHORELINE
35 FT. CHANNEL
ISLA SAN CARLOS
ISLA PAJAROS
ISLA ZAPARA
ISLA DE TOAS
ISLA PESCADEROS
Kilometers
0 1 2 3 4 5
0 1 2 3
Miles

FIG. 6.—LAKE MARACAIBO, VENEZUELA

Another such example is the entrance to the 18,000 sq km Lake Maracaibo in the western part of Venezuela (Fig. 6). A detailed description of the region of Lake Maracaibo is available.[4] When oil started moving out of the lake to the open sea, the draft of the vessels was limited to 9 ft by an outer and inner bar. Wave action is heavy on the 10 ft to 14 ft deep outer bar and the westward littoral drift is strong, while the normal tidal range is only approximately 1 ft. Surveys show that the channel over the outer bar migrated westward up to 3 ft per day and that a new channel would break through east of the migrating, deteriorating channel. The cycle of migration consumed a period of approximately 20 yr. In 1953 the Venezuelan government initiated a project to provide a 35 ft channel. This project was completed in 1956 but considerable maintenance dredging is expected because of the deep channel.

In Portugal the inlet at Figueira Da Foz in the estuary of the River Mondego (Fig. 7) is an example of an inlet in which natural bar by-passing has created satisfactory conditions for small craft for a long time.

Littoral drift along the Portuguese west coast is chiefly from north to south, and tidal range is approximately 10 ft. The inlet is being studied (1958) by the Hydraulics Division of the Laboratorio Nacional. The scope of the study[5] is as follows: (1) To investigate drifting conditions in the area of Cape Mondego and the adjacent beaches, particularly (a) to determine whether sand movement occurs or not from north to south beyond Cape Mondego, (b) to get an idea of the comparative intensity of drift in the foreshore and middle depth area, (c) to study the drifting pattern along Buarcos Bay, (d) and to estimate the speed with which sand coming from the sandy coast north of Cape Mondego reaches the mouth of River Mondego; and (2) to determine the conditions of passage of littoral drift material beyond the river mouth in a north-south direction.

Among other reasons the solutions of these problems were of great interest for verification of the accuracy with which natural conditions have been reproduced in the movable-bed model built to study jetty improvement of the River Mondego estuary and Figueira Da Foz harbor.

Because the area in question had a surface of some tens of square miles, and as there was no knowledge of the rate of littoral drift and the time required for the tracer material dumped into the sea to reach the harbor, it was impossible to use a shortlift radioactive isotope. The requirement had to be at least 90 days with maximum radioactivity. Four tons of iradiated AG 110 was chosen for the experiments, the result of which, as shown in Fig. 7 (figures indicating counts per minute), was as follows: 1. There is a considerable drift along Cape Mondego, 2. In the area of Cape Mondego only drift in the foreshore could be detected, as the difficulty of sampling (or measuring) prevented drift observations in the middledepth zones, and 3. It was verified that littoral drift material passed across the mouth of the River Mondego, but it was impossible to determine how this passage took place.

Improved Conditions.—Improved conditions include inlets that have been improved for navigation. Improvements must consider such installations as jetties, groins, dams, sluices, or similar structures that may help nature establish a better transfer arrangement without dredging operations.

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[4] "General Aspects of a Study on the Regimen of Lake Maracaibo," by G. A. McCammon, Coastal Engineering, No. 7, 1958, p. 625.

[5] "Tracing Sand Movement under Sea Water with Radioactive Silver,—Ag 110," Laboratorio Nacional De Engenharia Civil, Ministerio Das Obras Publicas, Portugal, 1958.

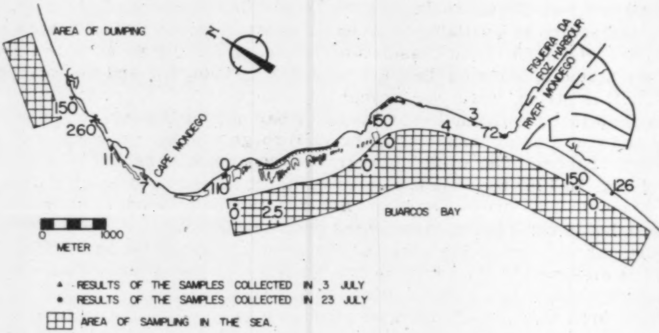
AREA OF DUMPING
N
FIGUEIRA DA FOZ HARBOUR
RIVER MONDEGO
CAPE MONDEGO
150
260
11
7
0
110
150
0
0
4
3
72
150
126
0
0
25
BUARCOS BAY
0
1000
METER
▲ RESULTS OF THE SAMPLES COLLECTED IN 3 JULY
● RESULTS OF THE SAMPLES COLLECTED IN 23 JULY
▦ AREA OF SAMPLING IN THE SEA.

FIG. 7.—FIGUEIRA DA FOZ, PORTUGAL

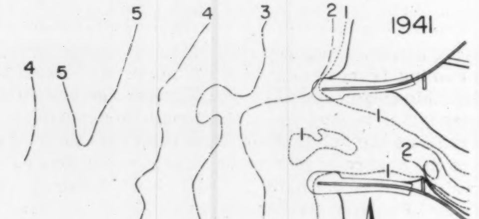
1941
5
4
3
2
1
4
5
1
2
1
N
(a)
4
3
2
1
1942
3
2
1
4
2
N
2
3
(b)
3
3
2
1
0
50
100 M.
DEPTHS IN METERS

FIG. 8.—THORSMINDE INLET, DENMARK

A small but interesting case has been in operation at Thorsminde (Thors Inlet) on the Danish North Sea Coast. Fig. 8(a) and Fig. 8(b) are charts of the inlet, which has navigation locks as well as locks for general flow. Tidal range is approximately 1 ft and wave action mostly heavy with 5 ft to 12 ft waves. Littoral drift is estimated to be at least 500,000 cu yd per yr. The inlet is protected by two jetties. Until 1944 both were 500 ft long.

During the period 1942 - 1947 two groins were built on the updrift side and the north jetty was extended 200 ft. The groins should catch excessive amounts of material migrating toward the inlet and the jetty-extension should push material to be by-passed farther seaward. Fig. 8(a) shows the situation on June 23, 1941, when the inlet conditions were particularly bad, with depths less than 3 ft between the jetties (normally 6 ft to 7 ft). It can be seen that there is no bar in front of the inlet. Fig. 8(b) shows the conditions on November 7 to 11, 1942. There is a bar, and at the same time the inlet conditions are good with depth of about 8 ft between the jetties. The problem of sanding up, which always takes place when the bar disappears, can be explained by the different distribution of littoral drift in a profile with a bar and a profile without a bar. At the first mentioned profile much sand by-passes on the bar where waves break. At the other profile most material migrates close to the shoreline, thereby shoaling the inlet entrance.

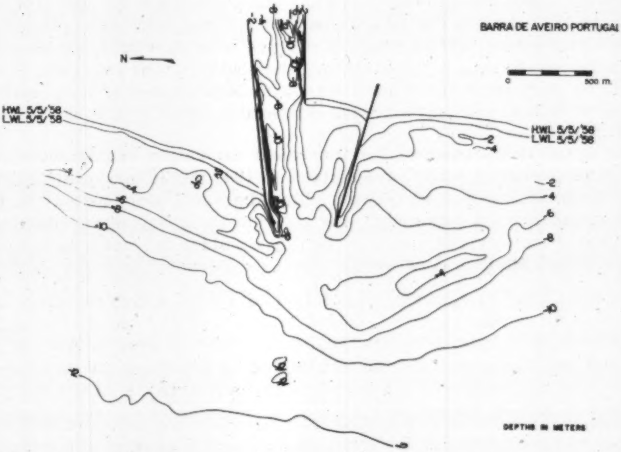

FIG. 9.—AVEIRO INLET, PORTUGAL

A similar situation exists with another inlet provided with sluices and located south of Thorsminde at Hvide Sande (White Sands). The depth on the bar that by-passes the material is usually 10 ft to 12 ft. Conditions are thus fair for navigation with fishing boats up to approximately 6 ft draft. Model experiments were conducted on this installation in the 1920's and was used for proper design of the entrance area and the jetties.

It is apparent from these two cases that by-passing occurs satisfactorily and without giving rise to leeside erosion, with a bar of a certain depth and width. However, in both cases, irregularities in the amount of littoral drift may momentarily result in shoaling of the respective inlets that must be cleared by flushing or dredging.

The Inlet of Aveiro on Portugal's Atlantic coast north of the River Mondego, (Fig. 9) as described by Carlos K. Abecasis[6,7] shows a similar development on a larger scale, but tidal flow resulting in a half-moon-shaped bar is of more importance.

The history of the inlet dates from the 10th century and shows a number of migrating as well as stable periods. Attempts to stabilize the inlet started in the 17th century but only one project, an artifically cut inlet in 1808 (designed by Carvalho), gave a long-range result. During the following period of approximately 140 yr, sand migrated on a bar across the inlet in a predominantly southward direction. During years of normal weather conditions the entrance being merely fixed by a jetty built along the southern bank extending to the low water line. Navigation in the inlet was considerable even if the configuration of the shoreline at the entrance was changeable and the original project was not well maintained.

After some time, to meet the increased navigation requirements, a new project was prepared, which was started in 1949 and is now practically completed. Detailed surveys, as mentioned by Abecasis,[7] demonstrate so far that littoral sands are not retained by the inlet on their way down-coast because the volume of sand expelled by the ebb tide considerably exceeds that brought in by the flood. He then concludes

> "And if so, the project being carried on correctly, solved the problem of assuring the depths required (indeed, more than required) in the channel with the least interference with the littoral drift regimen: in fact a strictly localized interference, both in time and in the space, as is also confirmed by the absence of any permanent erosion effects on the downdrift section of the sea shoreline."

A 12 ft to 15 ft shoal on the downdrift side may be important in the transfer of sand to the downdrift beaches when combined with heavy wave or rather swell action. Meanwhile, the present (1961) inlet situation is still so new that it may be too early to draw definite conclusions.

The Fort Pierce Inlet on the lower east coast of Florida (Fig. 10) should be mentioned here even though tidal currents play an important role in its natural transfer arrangement. Meanwhile, it is unlikely that sand would be transferred without the existence of a rather wide rock reef with 10 ft to 12 ft depths on the downdrift side of the inlet.

Investigations[8] conducted by the Coastal Engineering Laboratory of the University of Florida show that out of a littoral drift of approximately 200,000 cu yd

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[6] "The History of a Tidal Lagoon Inlet and Its Improvement," by Carlos Krus Abecasis, Coastal Engineering, No. 5, 1954.

[7] "Littoral Drift Problems in Portugal with Special Reference to the Behavior of Inlets & Sandy Beaches," by Carlos Krus Abecasis, Coastal Engineering, No. 6, 1958, p. 406.

[8] "Coastal Engineering Study at Ft. Pierce Inlet," by Coastal Engrg. Lab., Univ. of Florida, 1958.

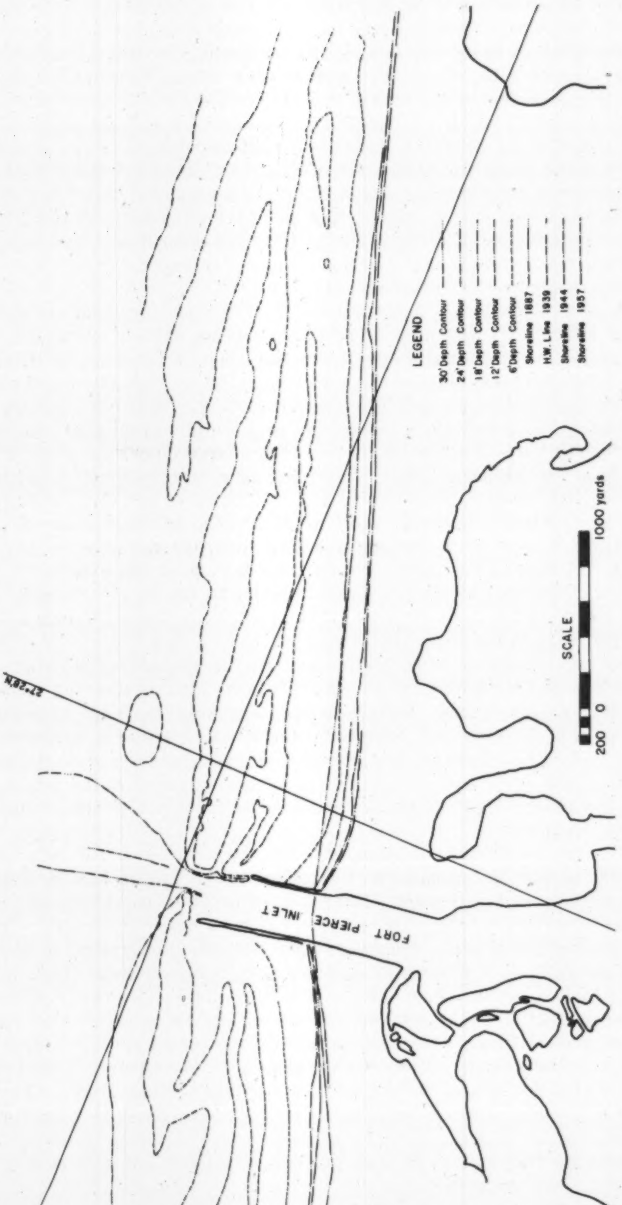
LEGEND
30' Depth Contour
24' Depth Contour
18' Depth Contour
12' Depth Contour
6' Depth Contour
Shoreline 1887
H.W. Line 1939
Shoreline 1944
Shoreline 1957
SCALE
200
0
1000 yards
FORT PIERCE INLET
27°28'N
80°17'W

FIG. 10.—FT. PIERCE INLET, FLORIDA

to 250,000 cu yd per yr only about 20,000 cu yd is deposited on bay shoals, 120,000 cu yd to 160,000 cu yd are jetted out in the ocean again by the strong ebb currents (5 fps to 6 fps). Part of the material jetted out, perhaps 40,000 cu yd, is lost to deep water. Another 20,000 cu yd is washed back into the inlet channel through the south jetty, and approximately 100,000 cu yd to 150,000 cu yd evidently given back to the downdrift side but apparently not until some few miles farther downdrift. Leeside erosion in the amount of approximately 100,000 cu yd per yr occurs in the first 2 miles to 4 miles south of the inlet according to the 1958 survey.

How this by-passing mechanism works is not known, but there is evidence that the flat rock reef with depths of 10 ft to 12 ft commencing at the extreme end of the 1,200-ft long downdrift (south) jetty and extending downdrift, plays an important role in this transfer action. It has been suggested that this question be further studied by radioactive tracing.

By-Passing At Harbors.—A special way of bar by-passing occurs at harbors on littoral drift coasts with weak or no tidal currents in their entrances.

An example may be found on the North Sea coast of Denmark at Hirtshals Harbor. A heavy littoral drift, perhaps 500,000 cu yd to 1,000,000 cu yd per yr, comes from the west with a strong longshore current. Part of the drifting sand is deposited in "tongues" along the updrift jetty while a great part passes the extended updrift jetty and deposits, by means of a large clockwise eddy current, in a shoal on the downdrift side. This shoal is gradually growing larger by deposits ranging between 50,000 cu yd and 200,000 cu yd per yr. Maintenance dredging is necessary in the 7 m to 8 m (25 ft) deep entrance channel to the harbor. The development in recent years shows decreasing depths on the downdrift shoal (10 ft to 13 ft). At the same time the shoal has extended farther downdrift with the result that the leeside shore is now being nourished from the shoal due to heavy wave and swell action, which apparently brings some of the material back to the shore.

An interesting example in giving nature a hand on by-passing is a suggestion by the NEYRPIC Laboratory in Grenoble, France, for transferring of sand at Port De Mucuripe in Brazil. Different laboratory experiments, as described in a report by G. E. Vincent,[9] have been conducted. A solution recommended is shown in Fig. 11. Material accumulated at the extreme end of the updrift jetty is supposed to be pumped across the harbor entrance to the outside of a smaller breakwater where it will be transferred downdrift by the gradual development of a barrier.

Another interesting case of by-passing sand at a harbor by natural action is found at the harbor of LaGuaira in Venezuela. This harbor has the entreme end of its nail-shaped updrift jetty located at 18 m (60 ft) depth. There is considerable littoral drift from east to west caused by heavy wave action (waves up to 20 ft from northeast). Some years ago a tanker ran aground midway out on the jetty at 30 ft to 40 ft depth and accumulated, in a short time, a great amount of sand behind it, demonstrating the heavy drift. Meanwhile, there has been no accumulation at the extreme end of the jetty and it is believed that the great depth and offshore bottom steepness may be responsible for this. Reference is, in this respect, made to Cornaglia's theory (Italy). Based on the experience with erosion of steep shores and gently sloping shores, Cornaglia claims that a neutral line or depth exists for any condition of wave action. Inside this depth material moves mainly onshore, whereas outside this depth it moves mainly away from shore due to the influence of gravity.

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[9] "Laboratory Study on Port De Mucuripe, Brazil," unpublished report by G. E. Vincent (Neyrpic, Grenoble, France).

Cornaglia's theory is not generally accepted but some model experiments performed[10] at the Massachusetts Institute of Technology Hydrodynamics Laboratory seem to indicate that the theory may be valid under certain circumstances such as are also demonstrated by nature.

Fig. 12 shows a project using breakwaters for the improvement of the fishing port of Scheveningen on the Dutch coast. The alinement of the jetties has been determined from model experiments intending to secure a maximum of by-passing of sand material and a minimum of depositing in the harbor entrance.

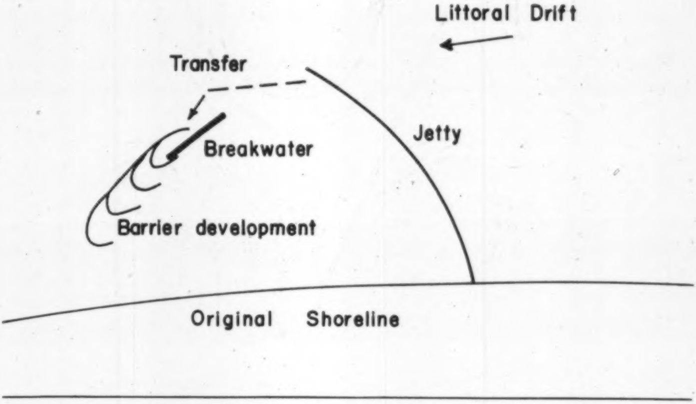

FIG. 11.—PORT DE MUCURIPE, BRAZIL

BY-PASSING BY TIDAL FLOW ACTION

The Principles Involved.—Where the ratio between the net littoral drift and the tidal flow $\frac{M_{mean}}{Q_{max}} = r$ is relatively small, the tidal flow usually plays an important part in successful by-passing of sand material.

Unimproved Inlets.—In general, sand transfer by tidal flow takes place in two different ways. These will now be examined.

Migration of channels and bars.—Tidal channels in inlets (particularly those running between the gorge and the ocean) are subject to migration. This means that they change location continuously, moving from one side of the inlet to the

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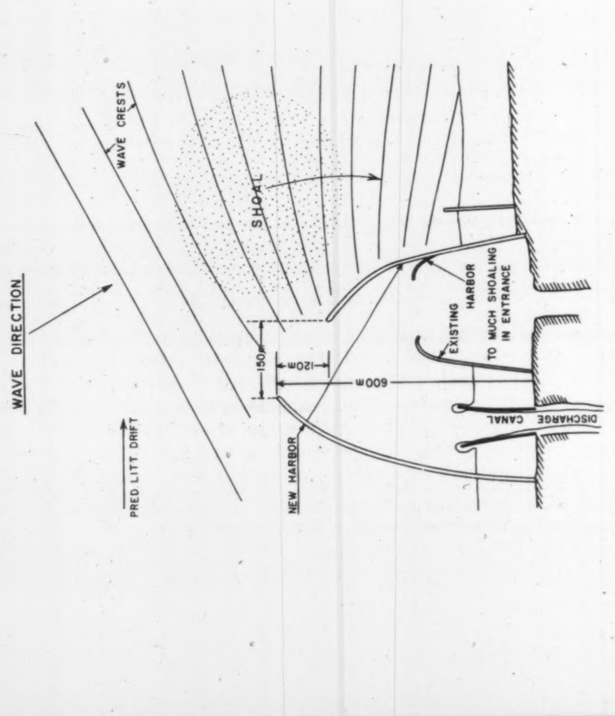
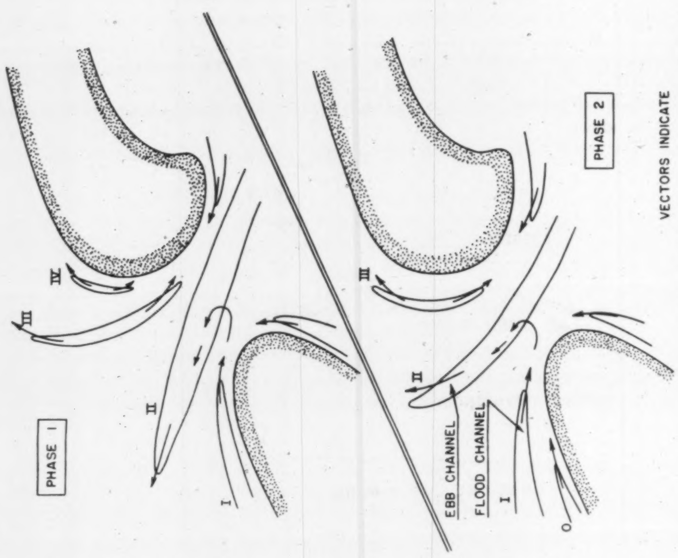
[10] "A Study of Sediment Sorting by Waves Shoaling on a Plane Beach," by A. Ippen and P. Eagleson, M.I.T., Hydrodynamics Lab., Tech. Report No. 18, 1958.


WAVE DIRECTION
WAVE CRESTS
PRED. LITT DRIFT
SHOAL
NEW HARBOR
150m
120m
600m
EXISTING HARBOR
TO MUCH SHOALING IN ENTRANCE
DISCHARGE CANAL

IMPROVEMENT SCHEVENINGEN HARBOR

FIG. 12.—PORT OF SCHEVENINGEN, HOLLAND


PHASE I
I
II
III
IV
PHASE 2
EBB CHANNEL
FLOOD CHANNEL
O
I
II
III

VECTORS INDICATE
RESULTANT DIRECTION OF SAND TRANSPORT
IN FLOOD AND EBB CHANNELS

FIG. 13.—EBB AND FLOOD-CHANNEL CHARACTERISTICS

other. In Fig. 14 this principle is demonstrated by phases 1 and 2 of a tidal channel system. Channels in phase 1 are numbered I, II, III, and IV. In phase 2 the locations of these channels have changed compared to phase 1 and a new channel "O" has developed along the updrift barrier. In this example the channels move from left to right and bars or shoals between the channels move in the same direction with the result that a bar occasionally joins the downdrift side beach where the sand is distributed along the downdrift coast by waves and littoral currents.

In most cases, migration of tidal channels takes place in the direction of the littoral drift. Sand is transported over the bar under the influence of waves and deposited on the updrift slope of the channels, thus forcing the shifting.

Because of the fact that tidal channels behave in the same way as channels in regular rivers, in curved sections sand motion along the bottom will have a component towards the inner curve. This, in turn, will increase the rate of migration of the channel in the direction of the downdrift (outer) side curve.

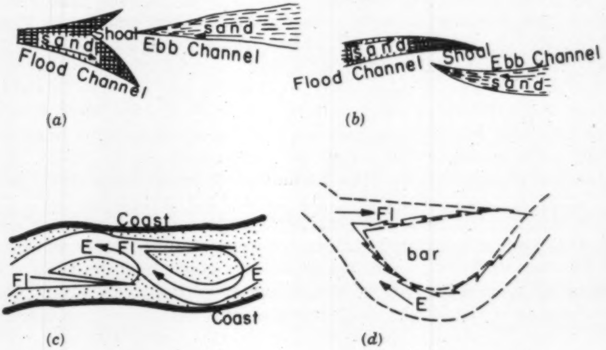


FIG. 14.—MIGRATION OF TIDAL CHANNELS

Transport of Sand by Tidal Flow in the Channels.—In the vicinity of tidal inlets, the (generally) strong tidal currents in the inlet change the littoral drift pattern entirely. Along the uninterrupted coastline wave action is generally the predominant cause for the transportation of material. In the vicinity of tidal inlets, however, transport of material takes place under the combined effect of waves and tidal currents.

In tidal rivers, estuaries and inlets channels can usually be identified as either flood or ebb channels. Flood channels carry predominantly flood flow, causing a resultant sand transport in a bayward direction. They, characteristically, have a shoal at their end. Ebb channels carry predominantly ebb flow and have a resultant material transport seaward and a bar or shoal at the end. Different types[11] of ebb and flood channels can be compared from Fig. 14. Fig. 14(a) shows the mutual evasion of flood- and ebb-channels by means of a forked tongue. Fig. 14(b) shows mutual evasion with flank attack of flood-and ebb-channels. Fig. 14(c) indicates the so-called circulating sand currents. The sand moves upstream in flood-channel, and downstream in ebb-channel. Fig. 14(d) indicates the true upstream and downstream movement of the sand in so-called circulating sand currents. A grain of sand may come back to its original place; dredging may be of small avail.

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[11] "Ebb and Flood Channels in the Dutch Tidal Waters," by Joh van Veen, Koninklyk Nederlands Aardryksundig Genootschap, Tweede Reeks, Deel 67, May, 1960.

It is remarkable that ebb and flood channels do not tend to be connected, but always leave a bar between them over which sand migrates. Fig. 13 shows, schematically, an ebb and flood channel system in an inlet area. Vectors indicate the resultant direction of sand transport in different channels. Sand particles make a zig-zag movement through the inlet from the updrift side to the downdrift side.

Jetty-Improved Inlets.—During the first phase of a jetty-improvement a great amount of sand is generally accumulated along the updrift beach. With an increasing amount of accumulation, by-passing of material gradually increases until a certain limit.

Fig. 15 shows three jetty-improved tidal inlets. In each case the ebb current has enough cleaning capacity to maintain the inlet channel by flushing action. Adequate by-passing in such cases needs considerable wave action to move the flushed material shoreward again.

The presence of a (special shaped) breakwater may develop an advantageous diffraction and refraction wave pattern for moving sand back to the beach. Compare this with Fig. 12 in which this is demonstrated for a harbor on a littoral drift coast in a movable bed model experiment.

At tidal inlets material is often accumulated on the inner (bay) shoals and this decreases the amount of material available for by-passing on the seashore. The inner shoal will usually not contribute much to the by-passing mechanism because it is normally located outside the area of appreciable wave action and strong tidal currents. It sometimes provides a convenient source for artificial by-passing by pumping of material from the shoal to the downdrift side such as recently suggested (band on model experiments) for the Bakers Haulover Inlet at North Miami (Coastal Engineering Laboratory, University of Florida).

Examples Of Inlets With By-Passing By Tidal Flow Action.

Unimproved Inlets.—An example of predominant tidal flow by-passing is the two groups of estuaries that form the southern and northern part of the Dutch coast.

Fig. 16 shows the configuration of the Dutch coastline in general. Predominant wave action from the southwest causes a predominant northward littoral drift although heavy storms may infrequently create high waves from the northwest.

Movement of sand is greatly influenced by the existence of comparatively strong tidal currents parallel to the coast.

The flood current runs in a northward direction and is the stronger. Its maximum velocity reaches 4 fps at approximately 1 mile from the shore. The southward ebb current is weaker. Its maximum velocity is approximately 3 fps.

The southernmost inlet, the Wester Schelde, forms part of the seaway to the port of Antwerp, Belgium. Its tidal prism is approximately 1,600 sq miles-ft and the cross-section near the entrance approximately 865,000 sq ft. The mean tidal range in this area is 10 ft to 12 ft.

Large quantities of sand are by-passed across this wide estuary by the tidal flow. However, the downdrift shore is insufficiently nourished, primarily be-

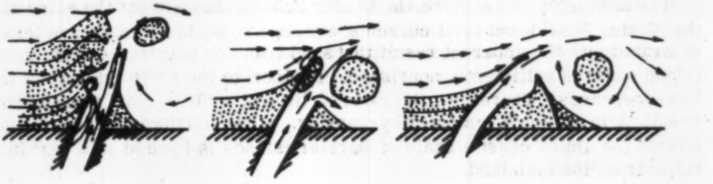

FIG. 15.—JETTY-IMPROVED INLETS

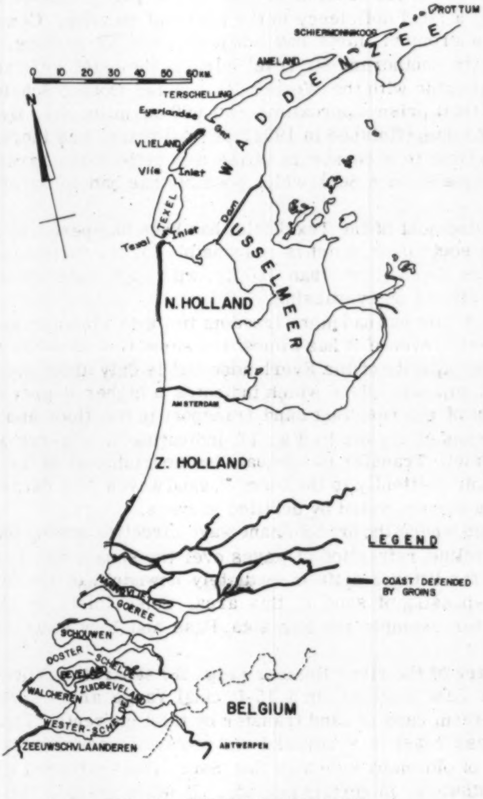
ROTTUM
SCHIERMONNIKOOG
AMELAND
0 10 20 30 40 50 60 KM.
TERSCHELLING
N
Eyerlandse Gat
VLIELAND
Vlie Inlet
WADDENZEE
TEXEL
Texel Inlet
Dam
IJSSELMEER
N. HOLLAND
AMSTERDAM
Z. HOLLAND
LEGEND
COAST DEFENDED BY GROINS
HARINGVLIET
GOEREE
SCHOUWEN
OOSTER-SCHELDE
BEVELAND
ZUIDBEVELAND
WALCHEREN
WESTER-SCHELDE
ZEEUWSCHVLAANDEREN
BELGIUM
ANTWERPEN

FIG. 16.—DUTCH COAST LINE

cause of a deep tidal channel close to the shoreline, which carries the material away.

The next inlet to the north, the Ooster Schelde, has almost the same size as the Wester Schelde but tidal currents, although equal in strength, are less concentrated with the apparent result that sand transfer is better and the downdrift island coast is sufficiently nourished. Farther to the north the size of the inlets decreases. Therefore, the ratio of littoral drift to tidal flow increases, resulting in an improvement of by-passing. At the northwest group of tidal inlets on the Dutch coast a chain of barrier islands is located at a varying distance from the mainland.

The area between the islands and the mainland is called "Wadden." The Wadden Sea is formed of tidal sand and mud flats, which are dry at low water. Numerous tidal channels exist. Tidal range is from 5 ft to 7 ft.

In various localities erosion occurs that is partly caused by tidal-channel shifting or by a local deficiency in the material balance. Generally speaking, however, this group of inlets has adequate sand by-passing. At the present time (1960) the southernmost, Texel Inlet, is the least satisfactory. The size of it is comparable with the Wester Schelde and Ooster Schelde of the southwest group (tidal prism approximately 1,600 sq-miles-ft). By construction of the Ysselmeer dam (finished in 1932) the tidal prism was increased by approximately 20% (due to a change in phase and reflection characteristics of the tidal wave in the Wadden Sea), which possibly has had an adverse effect on by-passing.

Free development of the Texel Inlet has been hampered by a heavy protection along its south bank, which is responsible for the formation of a deep channel (maximum depth more than 150 ft), with high concentration of currents and adverse effects on by-passing.

The inlet of Vlie has had more freedom in its development and presents adequate by-passing even if it has almost the same flow capacity as the Texel inlet. The flow capacity of the Eyerlandse Gat is only about one-fifth of the capacity of the adjacent inlets which indicates a higher degree of bar transfer. The direction of the resultant sand transport in the flood and ebb channels is shown by means of arrows in Fig. 17, indicating the zig-zag movement of the sand in the inlet. Transfer is adequate but nourishment of the downdrift shore takes place intermittently in the form of sand waves with decreasing amplitude downdrift, as demonstrated by detailed surveys.

In locations where the predominant wave direction makes only a small angle with the shoreline, refraction of waves over the shoals may lead to a reversed direction of the littoral drift immediately downdrift of the inlet, and prevent adequate by-passing of sand in this area. Some inlets on the west coast of Florida, as for example the Sarasota Pass and New Pass present this problem.[12]

The estuary of the river Humber (Fig. 18) separating Yorkshire from Lincolnshire in East England (in a 15-ft tidal range area) seems to present a unique long-term case of sand transfer by tidal currents. The littoral drift on the Holderness coast is southward and material accumulates at Spurn Head. Comparison of old maps indicates that Spurn Head extended itself up to 170 ft per year southward, in certain periods. In other periods the rate of advance was only 15 ft per year. Several times Spurn Head was cut off from Yorkshire. As mentioned by J. A. Steers,[13] Spurn Head undoubtedly does not represent more than a small part of the littoral drift accumulation from the Holderness coast. On the other side of the Humber, in Lincolnshire, there are large masses of sand at Donna Nook. These are probably not formed by the erosion of the Lincolnshire coast because there the native rocks are mostly warp and clay, without stones.

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[12] "Florida Coastal Problems," by Per Bruun, F. Gerritsen, and W. H. Morgan, Coastal Engineering, No. 6, 1958, p. 463.

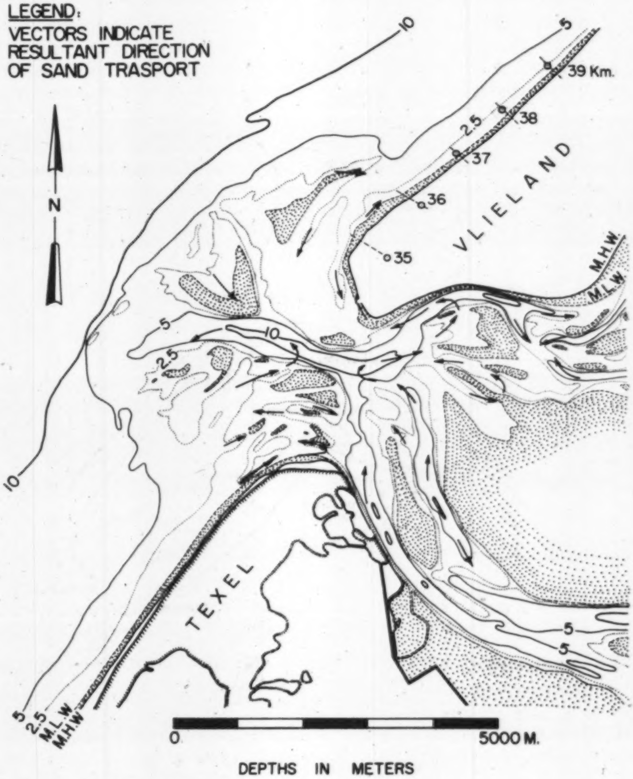
LEGEND:
VECTORS INDICATE
RESULTANT DIRECTION
OF SAND TRASPORT
N
10
5
2.5
39 Km.
38
37
36
35
VLIELAND
M.H.W.
M.L.W.
TEXEL
0
5000 M.
DEPTHS IN METERS

FIG.17.—RESULTANT SAND TRANSPORT VECTORS IN "EYERLANDSE GAT"

It is unlikely that anything coarser than sand would cross the Humber estuary, but the fact that Spurn Head has, at times, become detached from Yorkshire may explain the situation because, first detached from the mainland, the island may have worked its way across the river. Meanwhile there appears to be no record of this process in action.

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[13] "The Coastline of England and Wales," by J. A. Steers, Cambridge Univ. Press, London, 1946.

Whether a continuous transfer of sand by tidal currents across the estuary may take place still remains to be investigated, but the estuary has a rather advantageous configuration for such transfer.

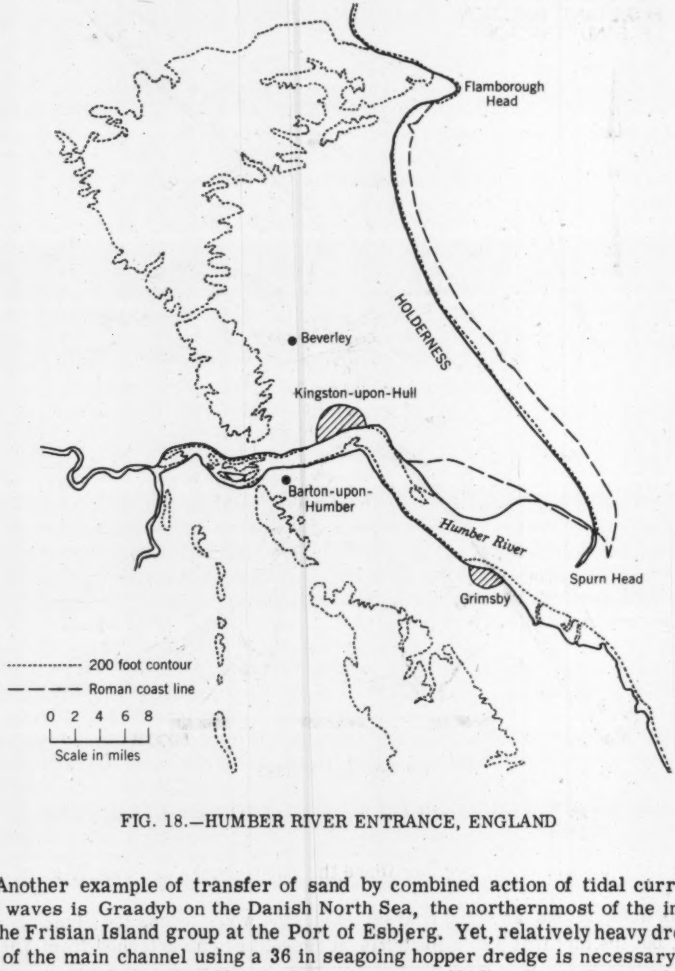
Flamborough Head
HOLDERNESS
Beverley
Kingston-upon-Hull
Barton-upon-Humber
Humber River
Spurn Head
Grimsby
200 foot contour
Roman coast line
0 2 4 6 8
Scale in miles

FIG. 18.—HUMBER RIVER ENTRANCE, ENGLAND

Another example of transfer of sand by combined action of tidal currents and waves is Graadyb on the Danish North Sea, the northernmost of the inlets in the Frisian Island group at the Port of Esbjerg. Yet, relatively heavy dredging of the main channel using a 36 in seagoing hopper dredge is necessary.

Improved Inlets.—Fig. 19 is harbor entrance at Abidjan, Ivory Coast, Africa. A cut was made to connect the ocean with a lagoon to accomodate vessels of

27 ft draft. Sand coming from the west is deposited by the flood current at M. The ebb current, which is strongly concentrated at that point, transports it in the direction of P, where part of it settles in a deep hole in the sea bottom.

Fig. 20 shows the entrance of Lagos Harbor, Africa. In order to counteract erosion on the downdrift coast model, experiments were conducted from which the following recommendations were made:

1. Construction of groin at C to counteract erosion of section BC;
2. Realinement of moles, with or without deposit of sand at D in the navigable channel. Currents and swell transport this from D to BC; and
3. Construction of a stationary sand pump at A to pump sand from A to B should also be considered.

Fig. 21 is the mouth of the Volta River on the Gold Coast, Africa. Sand is carried around the end of the west mole by flood currents and deposited at M. From there it is carried eastward by ebb currents that are concentrated at that point, because of the special alinement of the eastern mole. This sand is, for the most part, deposited in the downdrift area.

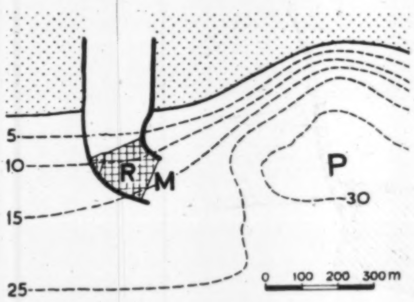

FIG. 19.—ABIDJAN HARBOR, AFRICA

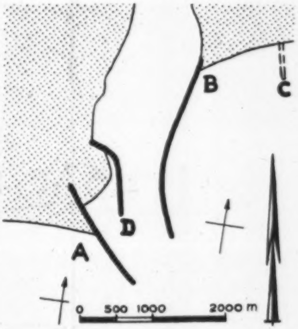

FIG. 20.—LAGOS HARBOR, AFRICA

The previously mentioned harbors at Abidjan, Lagos, and the Volta River entrance have all been investigated by hydraulic model studies in the Hydraulic Laboratory at Delft, Holland. In each of these cases, a satisfactory solution for sand by-passing was obtained.[14]

The jetty-improved entrance to the river Weichsel, Poland by-passes sand in the way that the river entrance has been designed and constructed for spoiling sand from the updrift side in the downdrift direction, depositing it temporarily on an offshore shoal where wave action carries it onshore again.

San Francisco Harbor has a large tidal prism (2,880 sq-miles-ft) flowing through the Golden Gate (875,000 sq ft). The strong tidal currents and heavy wave action are responsible for the huge half-moon shaped offshore bar with depths of from 12 to 18 ft. It is most likely that littoral drift material passes across the 50-ft-deep entrance channel but the process, perhaps a zig-zag movement of sand on the bottom, is not known. Downdrift side erosion obviously does not occur.

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[14] "History and Description of Hydraulic Model Investigations," Waterloopkundig Laboratorium, Delft, Holland, 1960.

By-Passing At Harbors.—The Zeebrugge Harbor in Belgium (Fig. 22) should be mentioned in this connection. This harbor, protected by a 4,500-ft long nail-shaped jetty was, for a long time, greatly bothered by silt deposits amounting to approximately 5,000,000 cu yd per year. The tidal range is approximately 12 ft and the tidal currents outside the harbor up to 5 fps to 6 fps. For some time the harbor was equipped with a 1300 ft opening (claire-voie) permitting tidal currents to flow through the harbor basin. This was unsatisfactory. Heavy deposits, mainly silt, continued and dredging of the deposits endangered the economy of the harbor.

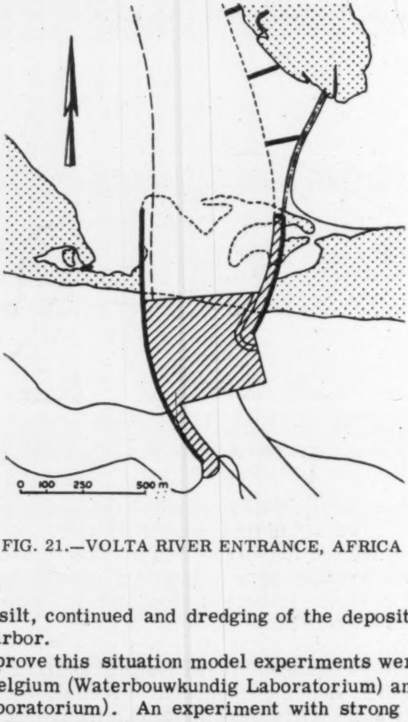

FIG. 21.—VOLTA RIVER ENTRANCE, AFRICA

In order to improve this situation model experiments were conducted after World War.II in Belgium (Waterbouwkundig Laboratorium) and in Holland (Waterloopkundig Laboratorium). An experiment with strong flood currents is shown in Fig. 23. The result of the construction of the big circular jetty on the shore-side was the elimination of a large silt-depositing eddy current in the harbor basin. The amount of silt deposits was reduced to less than 50%. The remainder of the material (mainly silt) by-passes the harbor with the tidal currents.

In model experiments with Karlsruhe harbor in Germany (river harbor) special jetty-configuration and design secured by-passing of heavy bed-load transport in the river flow.

ANALYTICAL EXAMINATION OF PROBLEM DESCRIBED

It is, thus, clear that natural by-passing depends on a number of different factors, such as littoral-drift magnitude and including longshore current velocity and wave action, quantity and velocity of tidal flow, material characteristics, and so on. These variables are not independent and depend themselves on other parameters such as shoreline and offshore bottom configuration and other geological and geographical factors.

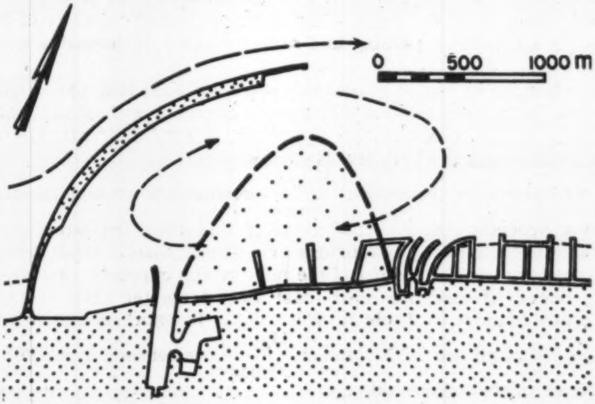

FIG. 22.--ZEEBRUGGE HARBOR, BELGIUM

As a first approximation those factors that seem to have a predominant influence on by-passing are considered, thereby ignoring those variables whose effect are probably of secondary importance. Although the material characteristics obviously are of importance in the by-passing mechanism, this parameter can be eliminated if considerations are restricted to inlets on sandy coasts with a grain size of 0.15 mm to 0.5 mm. Within this group the variation in grain-size material does not seem to have predominant effect on the by-passing procedure and mechanism.

Accordingly, the by-passing factor P is tentatively written as

$$P = f\left(\frac{\Sigma E_{wa}}{M_{mean}}, \frac{M_{mean}}{Q_{max}}, \frac{M_{max} - M_{mean}}{M_{mean}}\right)$$

in which P is the by-passing factor; E_{wa} denotes the wave energy of waves of a certain period from a certain direction; Σ indicates the total E_{wa} of all wave action; M_{max} refers to the quantity of maximum littoral drift per year; M_{mean} is the quantity of predominant mean littoral drift per year; and Q_{max} denotes the maximum tidal flow per second through the inlet. This equation is nonhomogeneous but useful for analysis.

The parameter $\frac{\Sigma E_{wa}}{M_{mean}}$ depends on the amount of wave energy and on the angle of approach of the waves with the shoreline.

Considering cases in which fair natural by-passing has been established at inlets (such as at Ponce De Leon Inlet in Florida; Oregon Inlet in North Carolina, which was recently dredged to from 12 ft to 14 ft; Thorsminde and White Sands in Denmark; and Figueira Da Foz and Aveiro in Portugal) under unimproved as well as improved conditions, it is characteristic that they all represent inlets with moderate to heavy wave action, causing much stir-up of littoral drift material. As a consequence of this littoral drift, it is strong with considerable suspended load transport utilizing the transport capacity of longshore currents. In all cases by-passing is a rather continuous operation without long interruptions.

With reference to Fig. 2 it can, therefore, be said that bar by-passing is possible whenever wave action and longshore currents, without being obstructed too abruptly by any configuration of natural or artificial nature, are able to build up a bar bridge to carry the sand transport.

The dimensionless parameter $\frac{M_{mean}}{Q_{max}}$ seems to be of significance for the by-passing procedure itself. The value of this ratio indicates whether by-passing is a predominantly bar-bridge or a predominantly tidal flow transfer, by which material is flushed out of the inlet by ebb currents carrying the material in downdrift direction or at least away from the inlet entrance area.

Reference is made to Table 1. When Q_{max} is expressed in cu yd per sec and M_{mean} in cu yd per yr, a value of $\frac{M_{mean}}{Q_{max}} = r$ between 5 and 900 has been found for the inlets considered.

From the experience about by-passing at those inlets, the following rule may be used as a guide:

$r < 10 - 20$ indicates predominant tidal flow by-passing

$r > 200 - 300$ indicates predominant bar by-passing

That M_{mean} is small compared to Q_{max} does not necessarily mean that conditions are advantageous or even ideal for tidal flow by-passing. A large Q_{max}, and therefore a smaller $\frac{M_{mean}}{Q_{max}}$, may still mean unsatisfactory by-passing if the tidal flow is not utilized properly for by-passing. Instead of flushing the material in downdrift direction it may be jetted out in deep water.

The parameter $r = \frac{M_{mean}}{Q_{max}}$ also plays a role in the stability of tidal inlets. The writers have previously noted[15] that in alluvial material the size of the gorge of the inlet under given boundary conditions depends on the size of the so-called stability shear stress τ_s along the wetted perimeter of the channel for the given conditions referring to springtide flow. The value of τ_s depends on various variables such as friction factors, bottom material, shape of cross section, wave action, suspended load and littoral drift characteristics, fresh water outflow and time history.[15]

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[15] "Stability of Coastal Inlets," by F. Gerritsen and Per Bruun, Transactions, ASCE, Vol. 125, 1960; and North Holland Publishing Co., Amsterdam, 1960.

TABLE 1.—FLOW AND LITTORAL DRIFT CHARACTERISTICS

INLET (1)	Q_{max}, in cu yds per sec (Springtide Conditions) (2)	M_{mean}, in cu yds per yr (Order of Magnitude) (3)	$r = \frac{M_{mean}}{Q_{max}}$ (4)
Amelandse Gat, Holland	36,000	10^6	28
Aveiro, Portugal	9,000[1]	$>10^6$	111
Big Pass, Fla.	720	10^5	139
Brielse Maas, Holland (before closing)	2,740	10^6	365
Brouwershavense Gat, Holland	30,000	10^6	33
Calcasieu Pass, La.	2,600	10^5	38
East Pass, Fla.	1,720	10^5	58
Eyerlandse Gat, Holland	19,000	10^6	53
Figueira Da Foz, Portugal	1,100	$>10^6$	910
Fort Pierce Inlet, Fla.	3,700	$1/4 \cdot 10^6$	68
Gasparilla Pass, Fla.	910	10^5	110
Grays Harbor, Oreg.	48,000	10^6	21
Haringvliet, Holland	25,000	10^6	40
Inlet of Texel, Holland	97,000	10^6	10
Inlet of Vlie, Holland	94,000	10^6	11
Longboat Pass, Fla.	1,430	10^5	70
Mission Bay, Calif. (before dredging)	1,130	10^5	88
Oosterschelde, Holland	100,000	10^6	10
Oregon Inlet, N. C.	5,100[1]	$3/4 \cdot 10^6$	147
Ponce de Leon Inlet, Fla.	1,450	$1/2 \cdot 10^6$	345
Port Aransas, Tex.	1,870	10^5	54
St. Augustine Inlet, Fla.	2,700	$1/2 \cdot 10^6$	175
San Francisco, Calif.	210,000	10^6	5
Scheveningen, Holland	sluices	$3/4 \cdot 10^6$	---
Thorsminde, Denmark	sluices	$1/2 \cdot 10^6$	---
Thyboron, Denmark	7,450	10^6	134
White Sands, Denmark	sluices	$1/2 \cdot 10^6$	---
Westerschelde, Holland	115,000	10^6	9

[1] increasing

In case of navigation inlet the value of $\frac{M_{mean}}{Q_{max}}$ should be kept low in an attempt to furnish sufficient depth in the channels across the outer and inner shoals. When the bottom of the inlet consists of rock the size of the gorge cannot adjust itself to its natural dimension in alluvial material. Velocities in such inlets may be much higher than at inlets in alluvial material and by-passing to downdrift side shores may for this reason be inadequate because of strong currents that push the material out in the sea beyond the littoral drift zone so that it is lost as nourishment material for beaches. An example of this is the Bakers Haulover Inlet on the southeast coast of Florida (Miami). Maximum current velocities reach 7 fps to 8 fps. Model experiments by the Coastal Engineering Laboratory of the University of Florida have indicated how important improvements for navigation as well as beach erosion including by-passing of material from the bay shoals can be secured.

The ratio $\frac{M_{max} - M_{mean}}{M_{mean}}$ indicates the littoral-drift irregularity which together with the $\frac{\Sigma E_{wa}}{M_{mean}}$ ratio and the absolute values of M_{max} and M_{mean} decribes the littoral-drift conditions.

The smaller $\frac{M_{max} - M_{mean}}{M_{mean}}$ is, the better conditions exist for satisfactory continuous by-passing under equal flow conditions. When this coefficient is large, the by-passing might still be adequate but irregular, with intermittent supply of sand to the downdrift beach.

Speaking of navigation, improved inlets may be designed to assist nature in by-passing by increasing the turbulences or by streamlined and less obstructive jetty improvements. If possible irregularities in the littoral drift magnitude should be equalized. At Thorsminde, Denmark (Fig. 8), groins have been built on the updrift side to regulate littoral drift. The updrift jetty has been extended to establish transfer in deeper water and drainage sluices have been built that are also useful for flushing the entrance if necessary. Navigation conditions are now rather satisfactory for boats of up to 6 ft draft and natural by-passing of an estimated quantity of about 500,000 cu yd per yr takes place.

At White Sands, Denmark, model experiments conducted 35 yr ago indicated the shape of an entrance that would be most useful for flow and for material by-passing, and sluices were built to regulate discharge and to be used in flushing. Navigation for boats with up to 8 ft draft as well as by-passing of approximately 500,000 cu yd per yr must be considered as generally satisfactory.

At Aveiro, Portugal, a long-term trial and error process resulted in a funnel-shaped entrance in which the curvature of the updrift jetty is particularly advantageous for by-passing. Model experiments with the inlet at Figueira Da Foz are in progress and a similar solution regarding inlet stability and by-passing is being sought. The present natural by-passing is reasonably satisfactory but greater depth for navigation is desired on the bar.

At a new harbor on the Danish North Sea coast at Hanstholm special measures for better by-passing are probably now secured by comprehensive model experiments and funds have been authorized to complete the harbor in accordance with the experiments.

No general formula or theory exists for proper design of inlets for by-passing, but if favorable natural conditions for by-passing exist this possibility can be tested by model experiments prior to construction. It will usually be difficult to avoid leeside erosion but improvements can be gained by using a streamlined jetty or guiding works on the downdrift side. This has proved favorable at several harbors.

If nature is not willing herself to establish by-passing, man may be able to help as demonstrated by model experiments with the harbor at Mucuripe, in Brazil, where a by-passing arrangement using long pipeline connections from one side of the harbor to the other is to be replaced by a smaller installation for pumping sand across the entrance of the port. A similar arrangement is under Examination[16] for Santa Barbara, Calif. in connection with construction of a downdrift breakwater or jetty.

In regard to inlets at which tidal currents play an important role in by-passing, the preceding paragraphs reveal that with the assistance of a considerable tidal flow there is a much better chance of developing a satisfactory by-passing arrangement if the flow is used in an intelligent way. The requirement is that deposits by flood currents in the inlet or entrance be removed by ebb currents and carried downdrift by wave action. With proper alinement of inlet jetties the situation can be favorably influenced so that adequate by-passing and sufficient depth for navigation go together.

Often, by-passing and navigation requirements are contradictory, particularly when bar by-passing dominates. Strong ebb currents at otherwise good navigation inlets with considerable tidal flow may be harmful by jetting material far out into the ocean where it is lost for the shore. If it is difficult to decrease inlet-current velocities, such inlets may be arranged so that material is deposited on bay shoals in such a way that ebb currents are able to carry the material back to the sea only to a minor extent. These bay shoals may then serve as barrow areas for artificial nourishment of the downdrift side by hydraulic dredging. Examples of this are Bakers Haulover and Hillsboro Inlets in Florida, at which material is pumped from bay or inlet shoals over onto the downdrift side. Sand traps built in the updrift jetty are also useful but often difficult to operate.

CONCLUSIONS

1. Analysis of the sand transfer at various inlets shows that in many cases adequate natural by-passing exists by which sand is transferred from the updrift to the downdrift side of the inlet.

2. Nature itself demonstrates two different methods of by-passing, namely bar by-passing, and tidal flow by-passing. Most cases of by-passing present combinations of these two general principles.

Whether we have predominant bar by-passing or predominant by-passing by tidal-flow action usually depends on the ratio between littoral drift and tidal flow.

If the predominant littoral drift (M_{mean}) is expressed in cubic units per year and the maximum tidal flow under springtide conditions in the same cubic units per second (Q_{max}), by-passing may be described by the ratio $\frac{M_{mean}}{Q_{max}} = r$

$r > 200 - 300$ indicates predominant bar by-passing

$r < 10 - 20$ indicates predominant tidal flow by-passing

The more regularly the transport of material by moderate to heavy wave action takes place, the better. Long periods with little wave action, and therefore little suspended load transport, may develop spits in or even barriers across the inlet by beach drift and bed-load transport. Continuous moderate to heavy turbulence seems always to be an advantage for adequate by-passing.

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[16] "Sand Bypassing at Santa Barbara, California," by R. L. Wiegel, Proceedings, ASCE, Vol. 85, No. WW2, May, 1959.

3. Natural by-passing occurs at unimproved as well as improved inlets. In the case of unimproved inlets, examples of adequate by-passing can be found with bar by-passing as well as tidal-flow by-passing, and with combinations of these.

Inlets with predominant bar by-passing are usually unfavorable to navigation. In most cases only small crafts are able to use them and improvement by dredging is often not economically justifiable.

When navigation requirements are associated with the desire for natural by-passing, inlets with tidal-flow transfer have the advantage over inlets with bar by-passing provided the flow is utilized properly.

With jetty-improved inlets, natural by-passing occurs in many instances on bars as well as by tidal-flow action. In both cases heavy wave or swell action seems improtant for transportation of sand back to the shore downdrift of the inlet.

Conditions for bar by-passing are more faborable in this respect when the resultant littoral drift is not too predominant. The contrary is true in case of tidal flow by-passing where a neutral direction of drift may result in continuously expanding offshore shoals.

Groins on the updrift side built for equalization of the littoral drift toward the inlet and sluices in the inlet useful for flushing of the channel have proved advantageous for proper bar by-passing at some places.

In the case of tidal-flow by-passing, the cleaning effect of the (often concentrated) ebb current and the desire for by-passing may be contradictory, yet optimum conditions for sand by-passing and navigation requirements may be obtained by proper alinement of the jetties securing spoiling of littoral drift material in the downdrift direction where waves may pick it up and bring it back to shore.

Where satisfactory by-passing conditions have been obtained, they have either been the result of a long-term (usually expensive) trial and error process or have resulted from hydraulic model experiments. In other cases, a compromise with nature has been obtained in such a way that material is allowed to be deposited in sand traps on the updrift side or inside the updrift jetty or on bay shoals from which the material is pumped over on the downdrift side.

4. In many cases of jetty-improved inlets or entrances to accomodate vessels of greater draft, by-passing arrangements using mechanical means such as dredging from stationary or non-stationary pumping plants are a necessity. Man is then faced with the following two problems, both related to inlet configuration:

1. The desirability of having an area well fitted for accumulation of sand coming from the updrift side and well fitted for pumping or dredging of the material accumulated; and
2. The desirability of having an inlet configuration which, at periods of maximum load of littoral drift material (when some material will escape the dredging or pumping plant), utilizes the inlet flow most adequately for flushing purposes, thereby possibly by-passing the sand deposited by the flood flow.

Regardless of the establishment of mechanical by-passing arrangements it should be remembered that an inlet first of all is a hydraulic-mechanism and the best inlets in existence are still those which were well conceived and well tested before they were actually born. Too many inlets failed because they were neither well thought nor tested before delivery.

DISCUSSION

J. R. BOWMAN,[17] **M. ASCE.**—The authors have made a significant contribution to the better understanding of littoral drift problems associated with the improvement of coastal inlets. Although a remedy for every case obviously is not within the scope of the paper, the recognition and classification of the natural processes involved offer clues to methods adaptable to many coastal inlets that heretofore have been considered hopeless.

Of the several inlets treated in the paper, the writer is most appreciative of the problems associated with the ultimate stabilization and improvement of Oregon Inlet in eastern North Carolina. This inlet is situated near the northeastern end of Pamlico Sound, a sound which has a surface area of nearly 2,000 sq miles, and which drains an area of some 25,000 sq miles in North Carolina and Virginia. The principal outlets to the Atlantic Ocean are provided by three relatively narrow and shallow inlets, one of which is Oregon Inlet.

The writer takes exception to the authors' statements regarding the longevity of this inlet. A 20-mile reach of sand barrier reefs, in which Oregon Inlet is situated, has long been regarded as "inlet prone"; it is not unlikely that the "English maps of the sixteenth century" depicted only the earliest recorded of several inlets, known to have opened and closed in this vicinity between 1585 and 1845. Other historical sources indicate that the present inlet was opened by a seiche generated in Pamlico Sound during the passage of a tropical storm in September, 1846; the name of the inlet is believed to have been derived from the destination of the first cargo schooner to utilize the newly opened passage.

Oregon Inlet has migrated some two miles southward since 1846, as noted by the authors. The erosion of its south bank has long ago engulfed the sites of the first Bodie Island lighthouse, the original Oregon Inlet lifesaving station, and a fort built by troops of the Confederacy during the Civil War. Although the present Oregon Inlet lifeboat station was built well south of the inlet, a scant mile now separates the two; considerable encroachment has taken place as the result of hurricanes and severe winter storms during the past decade.

The authors' stated tide range of 5 ft is somewhat misleading, inasmuch as it appears to include storm tides, and thereby precludes a proper differentiation between the effects of storm and normal tidal action on sand by-passing processes. Tide gage records for Oregon Inlet are indicative of a mean tide range of 2 ft and a spring tide range only a fraction of a foot greater. At Hatteras Inlet, one of the other principal inlets to Pamlico Sound, mean tide ranges of 2 ft outside and 0.5 ft inside the inlet have been recorded.

Inasmuch as the general arrangement of Oregon Inlet, in relation to its land and water environs, is not greatly dissimilar to that of Hatteras Inlet, it appears feasible to consider it as an inlet of the inadequate entrance type, in accordance with a system of inlet classification suggested by Caldwell.[18] Caldwell assumes that this type of inlet "is so inadequate that the tide range in the bay (sound) is only a small fraction of the tide range in the ocean." Moreover, "the time of strength of current in the inlet will tend to coincide with the times of high and low tide because the maximum hydraulic gradient will exist through the inlet at these times." Current velocities of about 4 fps have been measured in Hatteras Inlet under head differentials averaging 0.75 ft; it is not unreasonable to assume that velocities of similar magnitude occur in Oregon Inlet under normal tidal conditions.

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[17] Engr., Tidewater Constr. Corp., Norfolk, Va.

During stormy periods, the tidal characteristics of the two inlets are similar, but they differ in the direction of winds to which they are most responsive. Whereas Hatteras Inlet is particularly responsive to southeast and northwest gales, the winds that produce the greater head differentials at Oregon Inlet are the more frequent northeast and southwest gales. Critical winds generated by tropical storms can be expected to occur at both inlets once every 2 yr or 3 yr, on the average, but "hairy northeasters" are expected at least once each winter. The critical wind directions at Oregon Inlet are aligned with the longest fetch in Pamlico Sound, a distance of nearly 70 miles. During northeast storms at Oregon Inlet, heavy surf action is observed on the outer bar and along the shore, and tides at the entrance probably reach stages of 2 ft or 3 ft above normal; simultaneously, the waters of the sound are driven southwestward away from the inlet, exposing shoals that ordinarily are submerged by 2 ft or 3 ft. After the passage of a small hurricane in August, 1953, during which the sound was subjected to southwesterly gales for several hours, the writer observed high-water marks along the sound near Oregon Inlet estimated at 10 ft to 12 ft above normal sound levels. The steep hydraulic gradients and heavy surf action accompanying such storms have been responsible for the more aggravating occurrences of bank erosion and of shifting and shoaling of navigation channels at the inlet.

Oregon Inlet has been referred to as the gateway to Pamlico Sound and to the Outer Banks south of the inlet, a region that has become increasingly attractive by virtue of its commercial fishing and recreational opportunities. Exploitation of these potentials is hampered by the lack of adequate facilities for transportation into the region. Federal and state agencies have undertaken immediate remedial measures involving the dredging of a deeper channel through the inlet and outer bar (a project under way at this writing, and possibly the same one mentioned by the authors); and a bridge spanning the inlet to replace the present inadequate ferry system. The writer is not aware of any measures, in conjunction with either project, for preventing further migration of the inlet, or for ultimate stabilization of the navigation channel. The conclusions of the authors as to the instability of dredged channels as a sole means of improvement of certain types of inlets apply fully to Oregon Inlet; and the authors' predictions as to consequent events are verified by the amount of redredging that is required to complete the current channel improvement project.

PER BRUUN,[19] F. ASCE, and F. GERRITSEN.[20]—The authors appreciate Bowman's supplementary remarks to their statements about the Oregon Inlet in North Carolina. It is true that the inlet depicted on English maps from the 16th Century (located where today's Oregon Inlet is) does not carry the name Oregon, and an inlet in this area will probably always have a constant struggle for existence. Our tide figures are for spring tides as indicated in Table 1 and, according to the United States Coast and Geodetic Survey, Department of Commerce (USC & GS) the spring tides measure 4 ft to 5 ft on the open seashore, to which we were referring. This apparently was not made clear enough.

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[18] "Tidal Currents at Inlets in the United States," by Joseph M. Caldwell, Proc. Sep. No. 716, ASCE, Vol. 81, June, 1955.

[19] Head, Coastal Engrg. Labs., Univ. of Florida, Gainesville, Fla.

[20] Assoc. Prof., Coastal Engrg. Lab., Univ. of Florida, Gainesville, Fla.

TABLE 2.—FLOW CHARACTERISTICS OF OREGON INLET

Date (1)	Wind from (2)	Total flow on flood tide tidal prism (3)	Max. inst. flood flow, cu ft per sec. (4)	Total flow on ebb tide, acre feet (5)	Max. inst. ebb flow, cu ft per sec (6)	Max. velocities ebb fps. (7)	Max. velocities flood fps. (8)
Sept. 9, '31	N.E.	47,769	134,050	37,399	89,150	2.3	2.5
Aug. 31, '32	W.	42,726	129,100	40,054	102,700	2.6	2.4
Oct. 11, '32	S.W.	34,873	126,500	57,208	127,300	3.2	2.4

TABLE 3.—GORGE AREAS OF OREGON INLET

Date (1)	Cross section MLW, in square feet (2)	Cross section MHW, in square feet (3)	Source of Information (4)
1931-32	39,400	53,000	H. D. 310 81st Congr., 1st session
June 1958	43,300	54,400	C. of E.[a] maps
Jan. 1959	40,000	51,000	C. of E[a] maps

[a] U. S. Army, Corps of Engrs.

We share Bowman's interest in Oregon Inlet and have worked out some data that may be of interest.

Current velocities in Oregon Inlet were measured[21] in 1931 and 1932. From these measurements the volumes of the tidal prisms for flood and ebb were computed. The results are indicated in Tables 2 and 3.

It is apparent that winds have a strong influence on the quantity of flow through the inlet. Computation of the cross sections of Oregon Inlet compared with information of earlier date gives the figures indicated in Table 3.

The fact that the January, 1959 figure is somewhat smaller than that of June, 1958 is in agreement with the normal action of such an inlet. In the wintertime, littoral drift transport along the coast is generally heavier than in summer, so that more sand is entering the gorge of the inlet from the ocean tending to decrease the inlet's cross section.

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[21] House Document No. 310, 81st Congress, 1st session.

The cross-section stability of the inlet can be evaluated[22] by considering the $\frac{\Omega}{M}$ (tidal prism over annual littoral drift) and $\frac{Q_{max}}{M}$ ratio $= \frac{1}{r}$. With Ω = about 80×10^6 cu yd per half cycle we find $\frac{\Omega}{M} \sim 80$ and $\frac{Q_{max}}{M} \sim 5 \times 10^{-3}$ (1). Studies of numerous inlets have shown that those with a $\frac{\Omega}{M}$ ratio < 100 have a predominant transfer of sand on bars or shoals across the inlet entrance and comparatively small tidal flow; for this reason they may be rather unstable and are usually characterized by one or more narrow, frequently shifting, channels with rather high velocities. Oregon Inlet seems to be no exception in this respect.

In regard to the $\frac{Q_{max}}{M}$ ratio $\sim 5 \times 10^{-3}$, it has in a similar way been found that ratios $< 10 \times 10^{-3}$ represent a less stable situation and Oregon Inlet fits well into this picture also. Although Table 3 tends to indicate a rather stable cross-sectional area within the limited time period considered, it is quite evident that Oregon Inlet, with its high $r = \frac{M}{Q_{max}}$ ratio (215), its low $\frac{\Omega}{M}$ ratio (80) and low $\frac{Q_{max}}{M}$ ratio $\left(5 \times 10^{-3}\right)$, is not a very good navigation inlet.

With reference to remarks by Bowman this may explain why dredging has not been very effective. If a major lasting improvement is to be obtained, dredging will probably have to be carried out all the way through from deep ocean to deep bay. An improvement could, theoretically, be obtained by increasing Q_{max}, but a desirable increase to about twice the present value would cause considerable flooding in the bay area during storm tide conditions. The improvement would probably not be very stable unless it was accompanied by extensive jetty construction that, in turn, would cause a serious leeside erosion problem on the downdrift or south side.

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[22] "Stability of Coastal Inlets," by P. Bruun and F. Gerritsen, North Holland Publishing Co., Amsterdam, 1960.

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TRANSACTIONS

Paper No. 3277

ELECTRIC ANALOG MODEL OF A TIDAL ESTUARY

By H. A. Einstein,[1] F. ASCE and James A. Harder,[2] M. ASCE

With Discussion by Messrs. G. B. Fenwick; and H. A. Einstein and James A. Harder

SYNOPSIS

Five hundred miles of channel in the Delta Region, California, which are subject to sea water encroachment, may be protected by salinity barriers and a master levee system. An electric analog model developed at the University of California, Berkeley, Calif., predicts tidal amplitude and flows resulting from such modifications to the hydraulic system.

THE PROBLEM

Introduction.—The Delta Region, located about sixty miles east of San Francisco, occupies a strategic position in the geography of California, since it lies between the relatively arid San Joaquin Valley to the South and the Sacramento River Valley to the north, which has a developable surplus of water. The California Water Plan, a 25 yr to 50 yr blueprint for the development of the state's water resources, envisions that a substantial amount of water will eventually be transported southward across this general area. At the present time, the Tracy Pumping Plant, operated by the United States Bureau of Reclamation, Department of the Interior (USBR) is designed to divert 4,500 sec ft of water from the southern rim of the Region to the area west of Fresno, Calif.,

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Note.—Published essentially as printed here, in September, 1959, in the Journal of the Waterways and Harbors Division, as Proceedings Paper 2173. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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about a hundred miles further south. Eventually this rate will be tripled, by the installation of additional plants, to serve areas as far away as Los Angeles, Calif.

The entire region, embracing some 730 sq miles, was at one time an extensive marshy area where the two principal rivers draining the inland valley of California joined before they emptied into a series of bays and thence through the Golden Gate to the ocean. The peat soils of the region have proved very fertile when drained, and practically the entire region has been reclaimed into fifty or so islands, nearly all of which lie about 10 ft below sea level. In the process of reclaiming the land, some 500 miles of sloughs and channels were left, so that in 1959 the first impression is of a vast maze of waterways lacing the area.

At first sight, this maze of waterways would appear to provide an ideal means of bringing additional water southward, with perhaps only local improvements needed to enlarge the channel capacities. A more detailed examination, however, reveals a number of complicating circumstances. One is that a sea level channel to Stockton, Calif. on the eastern rim, must be maintained. The natural summer flow of the streams tributary to that region is not sufficient to repel all of the saline water which is moved by tidal mixing action in the channels leading from San Francisco Bay. Furthermore, the summer flow of the San Joaquin River contributes further to the salt content of the area, since it serves as the agricultural sewer of the northern part of the San Joaquin Valley.

These adverse salinity effects are being controlled to a certain extent at the present time as a part of the USBR operation of the Central Valley Project. Good quality water is released steadily throughout the summer from Shasta Reservoir on the Sacramento River in the northern part of the state. The flow is scheduled to maintain a steady flushing action in the channels leading from the Delta Region westward to the ocean. A flow of approximately 3,500 sec ft is required to keep the 1000 parts per million (ppm) chloride ion concentration point below the confluence of the two rivers near Pittsburg, Calif.

In the future, however, this water will be required for higher uses, and it will be necessary to segregate and prevent commingling of the higher quality conserved water from the northern part of the state with the lower quality, higher salinity water that now enters the Delta Region from the sea and from agricultural drainage. The accomplishment of this task will involve major changes in the channel system and consequent alterations in the pattern of the tides in those channels which are still open to the influence of the sea. Although the details of a practical plan for the Delta are being worked out (as of 1959) by engineers of the California Department of Water Resources, several definite features are emerging. These are as follows:

1. Salinity barriers will be placed in the upper parts of the Sacramento River System so that the major part of flood flows from the Sacramento Valley can continue to pass unimpeded from the Yolo bypass through the lower Sacramento and into Suisun Bay (see Fig. 1). Surplus water from the northern part of the state, carried by the Sacramento River, will be diverted southward at this point.
2. Diverted fresh water will be passed under the Stockton Deep Water Channel by inverted syphons; the shipping channel will be isolated from the fresh water channels and will be open to the sea. It will continue to serve as a floodway and as a drain for poor quality irrigation return flows and other waste water.

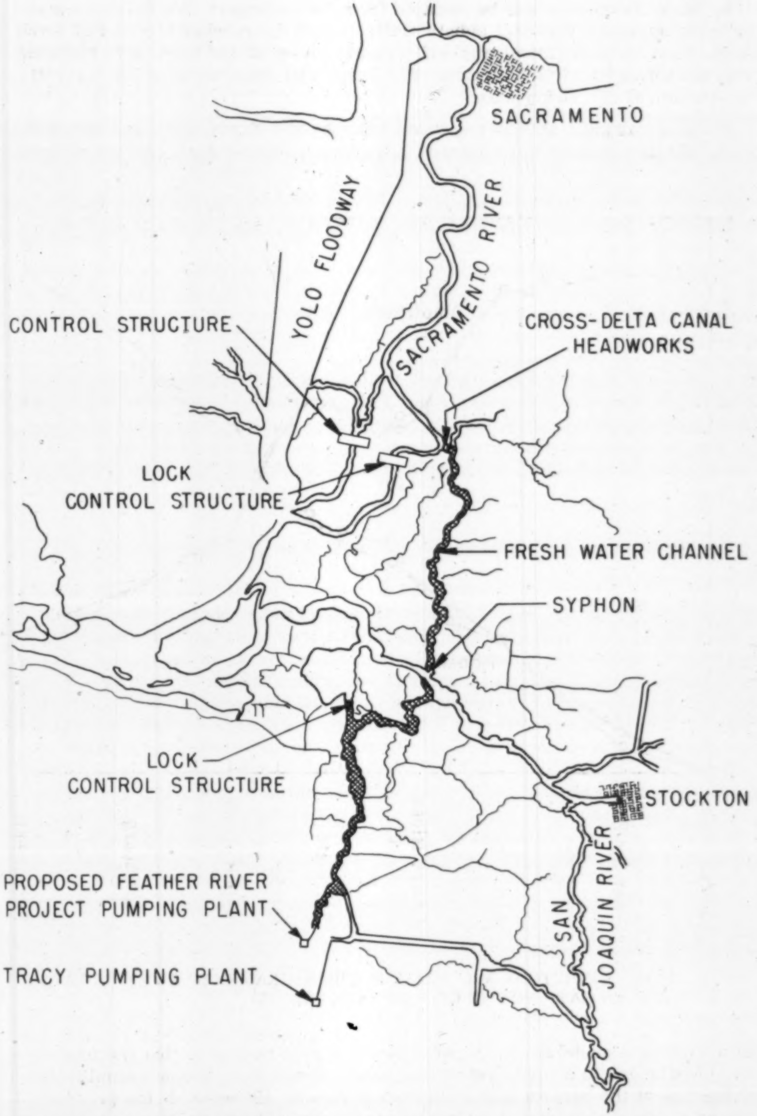
SACRAMENTO
YOLO FLOODWAY
SACRAMENTO RIVER
CONTROL STRUCTURE
CROSS-DELTA CANAL HEADWORKS
LOCK
CONTROL STRUCTURE
FRESH WATER CHANNEL
SYPHON
LOCK
CONTROL STRUCTURE
STOCKTON
SAN JOAQUIN RIVER
PROPOSED FEATHER RIVER PROJECT PUMPING PLANT
TRACY PUMPING PLANT

FIG. 1.—ONE OF SEVERAL PLANS FOR IMPROVEMENTS IN THE SACRAMENTO-SAN JOAQUIN DELTA REGION (MODIFIED BIEMOND PLAN)

3. Major floodways will be isolated from the secondary channels by a master levee system. This will substantially reduce the number of miles of levee which must be maintained and will remove most of the secondary channels from the influence of tidal action. The latter will continue to be used for the distribution of irrigation water.

From a hydraulic standpoint, these changes are substantial, and normally one would not proceed very far with any plans involving such changes without first making a model study. A particularly cogent reason is that the introduction of barriers in a tidal system can easily result in a resonant amplification of the tide at the barrier and downstream from it. However, in the present instance a reasonably scaled hydraulic model would be very expensive because of its sheer size. At a horizontal scale of a thousand to one, which is the scale

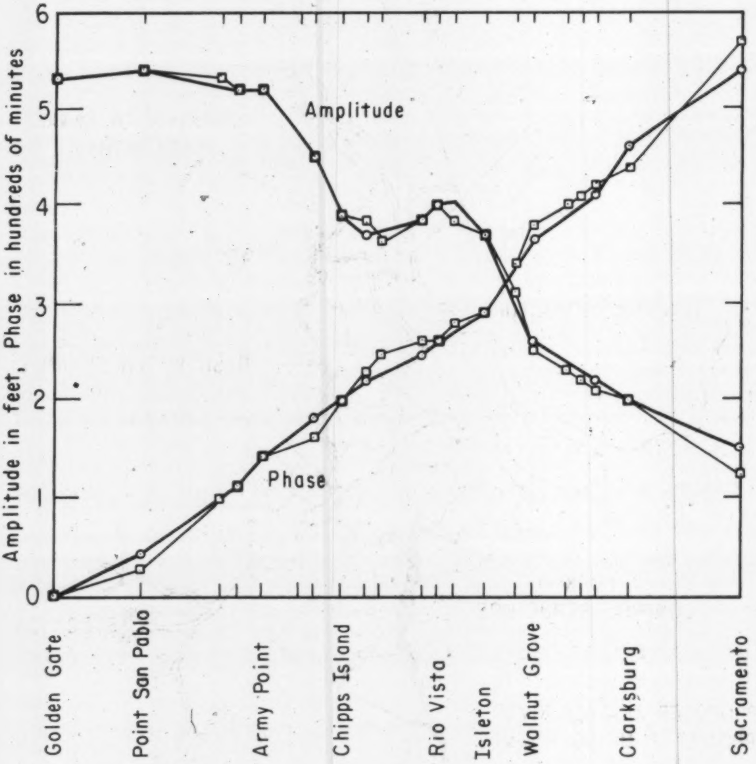

FIG. 2.—TIDAL AMPLITUDE AND PHASE IN THE SACRAMENTO RIVER FROM THE GOLDEN GATE TO THE CITY OF SACRAMENTO

used by the Corps of Engineers for the San Francisco Bay model, the area covered would be about four hundred feet on each side and would equal that of three football fields. Such a model would have to include not only the Delta Region proper, but also the entire San Francisco Bay, because it is necessary that reflected waves due to the tide be allowed to traverse the entire system from the ocean outlet to the farthest upstream area of interest. The model must be constructed with the same scale factors throughout, and represent with the same scale the 4 million cfs peak tidal flow at the Golden Gate and flows as small as 1,000 cfs in some of the upstream channels.

It was with this problem and with a severely limited budget of time and money that the State of California Department of Water Resources approached a staff member at the University of California, Berkeley, Calif., to see if there was any chance of finding analytic solutions. In a previous study, two members of the University staff had made a series of analytic calculations for the prediction of tidal amplitudes below various proposed barriers in San Francisco Bay proper.[3] However, the geometry of the upstream channels is so complicated by branching that no analytic treatment seems possible. Instead, the staff member suggested that an electric analog model might give some of the answers needed for a relatively small cost, and that some work along these lines was already being done in the University's hydraulics laboratory. After further consultation, a service agreement was made with the University for the construction and operation of an analog of the Delta Region. The agreement period extended from March 1, 1956 to June 30, 1957.

THE ELECTRIC ANALOG MODEL

What is an Analog Model?—From a fundamental standpoint, one physical system can be considered an analog, or model, of another when it is governed to a satisfactory degree of approximation by the same equations as is its prototype. In this sense, a hydraulic model is an analog of the full scale structure it represents. Furthermore, the electric analog models to be described bear a remarkable resemblance, in their adjustment and operation, to hydraulic models. The term analog models is used to avoid a confusion with analog computers, which are machines for directly solving ordinary differential equations.

From a slightly less fundamental standpoint, it can be said that two systems are analogous when we observe the same kinds of action in each. Thus, a very long wire can transmit electric waves along its length in a fashion very similar to the way a long canal will transmit a train of long period waves along its length. Furthermore, at the open end of the wire the wave is reflected in the same way as at the closed end of a canal.

Here, as in the analogs to be described subsequently, the voltage level is the analog for water surface elevation and electric current is the analog for discharge rate.

In this example the action is the same, but the equations are only the same to a first approximation. In order to make the best use of such an analog for the study of hydraulic systems, the fundamental equations must be examined to see in which ways they are similar, and in which ways they differ. The main difference, in the case of tidal flow analogs, is in the way energy is dissipated. If the electric circuits can be altered in this regard to correspond more closely to the hydraulic system, it can be shown that it is possible to neglect some of the other differences, or to at least make corrections for them.

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[3] "Report on the Study of Tidal Stages and Flows as Affected by the Construction and Salt Water Barriers in San Francisco Bay," by H. A. Einstein and R. A. Fuchs, Dept. of Water Resources, State of Calif., 1955.

One difference is that in a canal the momentum depends on the depth as well as the flow rate whereas the corresponding property of an electrical circuit, the strength of a magnetic field, depends only on the electrical current. The practical effect is that a type of non-linearity which distorts the tide-induced waves is not present in the electric analog. Its magnitude depends on the wave-amplitude to depth ratio. However, in those cases where this ratio becomes appreciable, the friction has usually increased to the point where it predominates over even the linear portion of the inertia, so that from a practical standpoint the most important non-linearity for tidal flows and river flood waves is due to friction.

The friction, like the inertia, depends on the water depth. Furthermore, because there is only an electric analog quantity for discharge, and none for velocity, its value must be expressed in terms of the discharge before conversion into electrical terms. If q is the discharge per unit width and y is the water depth, the friction slope is given by an arrangement of the Chezy equation:

$$S_f = - \frac{|q|q}{C^2 y^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Fig. 1 shows (by the absolute value notation) that the sign of the slope is always such that the flow is opposed, and that there is a strong dependence on the water depth. The quadratic form, apart from intensifying the friction in regions of high flows, introduces a symmetrical distortion to the waves. Without the dependence on depth, the distortion would affect positive and negative parts of the wave equally, generally "clipping" the maximum flow peaks. However, for progressive waves, an upstream flow is associated with greater depths and ebb flows with smaller depths. Even if the discharge and surface displacement are not exactly in phase (phase is here defined as the relative displacement of the maxima), there will be a net reduction in the friction for upstream flows, while downstream (ebb) flows encounter increased friction. Lacking an exit for flows at the upstream end, a compensating increase in the average water level will occur with distance upstream. This is observed as an increase in the "tidal plane" elevation and is found in estuaries whether there is a superimposed river flow or not.

The overwhelming importance of the friction contribution to the non-linear characteristics of the hydraulic system led to the development of a special electrical unit which simultaneously simulates the square-law characteristic and the depth dependence of the friction. The battery operated transistor circuit is described in more detail in Appendix I. It operates for current flow in each direction, and is relatively simple. Over twenty were built for incorporation in the Delta Analog.

A third way in which the two systems differ is that the properties of the electrical transmission line must be lumped into discrete elements of inductance, resistance, and capacitance. That is, the river channels must be broken into reaches, each of which are then simulated by a set of electrical elements. If these reaches are too short, there is a waste of effort in building too many elements. However, if they are too long relative to the length of the waves which must be transmitted, the higher frequency components will be attenuated artifically relative to the lower. Of more importance, long before the higher frequencies are seriously attenuated they suffer a phase shift which is not correctly related to that of the lower frequencies. A more extensive treatment of the errors due to the previously-mentioned non-linearities is available.[4] The errors due to lumping are treated in Appendix II.

A fourth point is that there is no electrical equivalent of kinetic energy and thus no way to include the $V^2/2g$ term in the equation of motion. The magnitude of this term relative to that of the temporal acceleration is given by the Froude number, and is thus small in most tidal estuaries. A partial remedy for finding the actual depth at a given point is to compute $V^2/2g$ and to correct the indicated water surface elevation by this amount. This will lead to a change in the cross sectional area, but this introduces only a second order correction to the other terms.

In spite of these differences, it has proved possible to obtain results from the analog closely duplicating the existing flows in the Delta. What is indicated is that the operation of an analog model requires at least as great a knowledge of hydraulics as does the operation of a regular hydraulic model. In the latter most of the above-mentioned diffifulties are painlessly looked after by the model itself, but in the case of the analog the operator must undertake this responsibility. He must be able to abstract the properties of the hydraulic system and translate them into electrical terms, and vice-versa. On the whole, however, there is a striking similarity between hydraulic models and analog models, from the standpoint of construction and operation, that lends some familiarity to the latter.

Analog and Hydraulic Models Compared.—The steps taken in the Delta Analog Study paralleled in many ways those taken in a more conventional investigation, and perhaps it will be helpful in explaining the newer method to compare it wherever possible with the more familiar. The first step in each case is to gather sufficient information about the prototype channels so that they can be duplicated in the laboratory—in the one instance in the form of miniature channels, and in the other in the form of suitable electric circuit elements.

The technique of constructing a reduced scale model of the prototype channels is straightforward, if somewhat laborious. Usually a considerable amount of bottom detail is included, and perhaps it is only in this way that some details of the flow, such as velocity distributions, can be duplicated. This type of information cannot be obtained from an analog model, which duplicates the wave transmission characteristics of the channels and gives only the velocity averaged over the cross sections.

Preliminary to the construction of the analog model the prototype channels must be divided into reaches within which the cross section does not change too much and which are short enough for the purposes of the investigation. For the investigation of tidal flows, this length is about 5 miles, and for flood waves it can be 25 miles or more. The wave transmission characteristics of this reach depend on the average cross sectional area, the water surface area, and the friction. The first two are obtained from field data, as in the case of hydraulic models, but the friction must sometimes be determined by trial, as is nearly always the case with hydraulic models.

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[4] "Non-Linear Tidal Flows and Electric Analogs," by J. A. Harder and F. D. Masch, Journal of the Waterways and Harbors Div., Proceedings, ASCE, November, 1961.

In the construction of the Delta Analog, the electrical components were assembled for each channel reach into a small plug-in unit. Each contains a variable inductor, a variable resistor for use in introducing linearized friction, and a variable capacitor. The relations between the hydraulic parameters and the values of time scale, capacitance, inductance, resistance, and so on, that must be used, are well described elsewhere so that it is sufficient to remark that the value of capacity is directly proportional to the water surface area, and the inductance is related to the cross sectional area and the length of the reach. The plug-in units are adjusted to the indicated values of inductance and capacitance on a special test panel, and the resistance is set according to the best estimates of the friction. The resistance value is further adjusted in the verification procedure or when square-law friction elements are introduced.

The sockets to receive these units are installed on a large table, and are interconnected by wiring on the under side. After the adjustment of the plug-in units they are inserted in their sockets and the analog is ready for the second stage, that of verification.

In a hydraulic model, the corresponding steps are the physical construction of the model channels. If the model has been built to an exaggerated vertical scale, as is usually the case, the model friction must be increased in the same proportion and this is often done by inserting copper strips into the bottom. In the process of verification, these strips can be bent down, or straightened up, until the tides of some historical period are satisfactorily duplicated. During the verification stage of the operation of an analog model, the friction is adjusted in a similar way, except that the adjustment is done with a screwdriver on the individual units. At this point the hydraulic model is in most instances ready for the active phase of the investigation, but one additional verification step is usually necessary in the case of the analog, to insure that the correct cross sectional areas have been introduced. The inductance value should be proportional to the length divided by the average cross sectional area, or more correctly the square of the length divided by the total volume of water in the reach. The part of the volume contained in shoal areas that does not contribute appreciably to the inertia, because of its low velocity and isolation from the main stream, should be excluded from the determination. Although it is not always possible to determine at first the correct amount to exclude, if the correct value is chosen, the velocity of wave propagation in the prototype channel will be duplicated in the analog.

The results of one verification run are given in Fig. 2. The amplitude (peak to trough) and phase (time lag) of the tide in the channels leading from the Golden Gate to Sacramento, a distance of one hundred miles, are shown by the circled points. These measurements were made on the neap tides of September 13, 14, 1954, for which good records are available. The results from the analog are marked with squares.

Tidal amplitude could be measured to 0.05 ft on the analog and time to the nearest 10 min. The field data are determined to about the same accuracy. Almost all of the discrepancies are explained in this way. The analog measurements of tidal plane elevations were too low by 0.25 ft at the junction of the two rivers. This is accounted for by the salinity gradient. No correlation could be found between the daily variations of the tidal plane there and wind direction and velocity at Berkeley, 30 miles to the southwest.

Tide Generation and Measurement.—Both neap tides and spring tides were simulated; the corresponding voltage waveforms were generated at low power level and then introduced at a point corresponding to the bar outside of the Golden Gate by a low output impedence amplifier. The neap tides were represented by a 5,000 cycle per sec sine wave and the spring tides were generated with a photo follower - masked cathode ray tube device. There does not seem to be very much similarity to hydraulic models here, but the combination of float actuated pumps and cam operated surface elevation controls used in hydraulic models is actually a servo-mechanism with feedback, and corresponds in a theoretical way to our use of heavy feedback to insure a low output impedence for the tide waveform amplifier.

In other ways, too, there are theoretical similarites. When it is desired to terminate a hydraulic model at an upstream point, an artificial impedence is introduced in the form of some kind of energy dissipator plus basin storage. Similar combinations of resistive and reactive impedence can be used in an analog model for the same purpose, but with considerably greater ease.

The electronic equipment for measuring amplitudes, flows, and time intervals in the analog were specially designed. All links in the measuring chain were direct-coupled, allowing the steady state value of voltage or current to be measured together with the alternating, or tidal, fluctuations. Thus, a zero level could be fixed on the cathode ray tube display and the absolute value of the flow determined.

Two probes were used in obtaining information. Each could be switched to measure either current or voltage, and further switching allowed any two of these four quantities to be displayed simultaneously (except that no provision was made for simultaneous current measurements). Furthermore, by the use of a calibrated delay line, the phase difference or time lag between the two measurements could be determined.

The two selected signals were combined in an electronic chopper and appeared together on an oscilloscope screen, where amplitudes could be scaled from the graticule. The system was calibrated frequently by applying known voltages (from secondary standard mercurous chloride cells) to the probes; thus the accuracy was not dependent on the gain stability of the electronic system, though this was surprisingly good.

Results.—Since the Department of Water Resources has not completed (as of September, 1959) their salinity barrier investigation, it would be premature to indicate at this place the actual results of the study, as well as their engineering significance. It may be most interesting to the reader on the other hand to learn what kind of results may be obtained.

One of the most important questions to be answered by the analog is that of predicting tidal elevations if the channel system is changed by barriers placed into various channel sections, by cut-offs between channels, by enlargement of individual channel sections or by any combination of such changes. Simultaneously, the flow velocities are obtained from which it is hoped that the extent of the expected salinity intrusions may be derived. It has become very clear from the past results that a barrier does not always reduce the tidal prism, but often may cause the tidal flow to be reflected back into the channel system, locally increasing the tidal heights. Such reflections must be reduced to a minimum by the proper location of the barriers.

Similarly, the opening of channels may increase the tidal effects in some places, reducing them simultaneously in other sections. The analog is here an extremely valuable tool in balancing these effects such that undesirable increases of tidal heights may be prevented both in the final plan and in the temporary stages of construction.

As of September, 1959 the analog has been moved to Sacramento, Calif., where the Department of Water Resources plans to use it in a continuing study of fresh water flow distribution and to schedule the construction so that closures can be made in the easiest way.

APPENDIX I.—THE TRANSISTORIZED SQUARE-LAW RESISTOR

During the investigation, a new type of square-law resistor was developed which should allow a considerable advance in the technique of applying electric analogs to open channel flow problems. Some of its features are as follows:

1. For a given voltage drop, the current can be adjusted over a range in excess of twenty to one. This allows the same type of unit to be used to simulate large and small channels.
2. For each setting, the voltage drop bears a square-law relationship to the current except for small values of current. This simulates the variation of energy loss with discharge in open channels.
3. For a given voltage drop, the current can also be made a function of the voltage existing between the unit and ground. This simulates the variation of discharge, for a given water surface slope, with water depth.
4. The units are small, operate indefinitely from self-contained batteries, and are low in cost.

Theory of Operation.—For steady flow, the discharge in an open channel, according to the Manning formula, is related to the water surface slope, S, the cross sectional area A, and the hydraulic radius R, in the following way:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \quad \text{. . . . .} \quad (2)$$

in which n is the Manning roughness coefficient. Assuming n to be constant for a given channel section, the factors $\frac{1.486}{n} A R^{2/3}$ can be represented as a function of the elevation z above a horizontal datum; the equation can then be written as

$$Q = F(Z) S^{1/2} \quad \text{. . . . .} \quad (3)$$

Similarly, the magnitude of current, i, in the square law unit, can be expressed by the following formula relating it to the voltage e above ground and the voltage drop, Δe, across the unit:

$$i = f(e) (\Delta e)^{1/2} \quad \text{. . . . .} \quad (4)$$

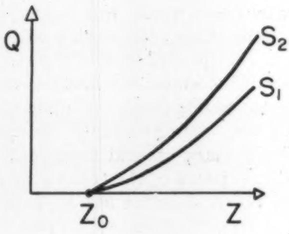

FIG. 3.

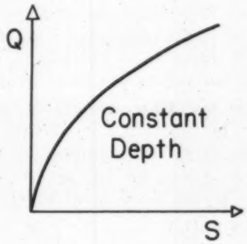

FIG. 4.

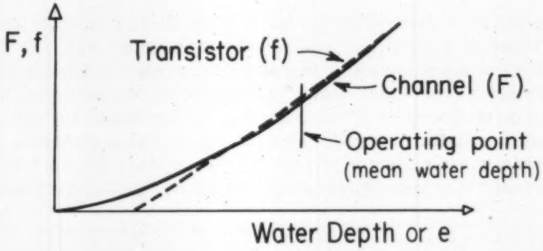

FIG. 5.

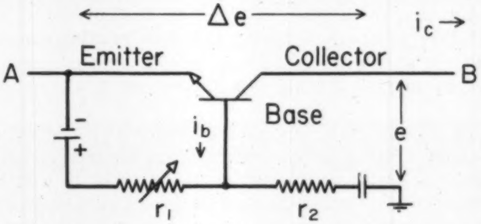
Δe
i_c
Emitter
Collector
A
B
Base
i_b
e
-
+
r_1
r_2

FIG. 6.

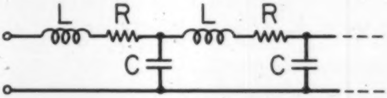

FIG. 7.

These relationships can be shown graphically in two ways. For a given value of slope S, the discharge - water surface elevation relationship will take the general form shown in Fig. 3, in which z_0 is the elevation of the channel bottom and S_2 and S_1 are two different values of the slope. Alternatively, for a particular elevation, the discharge-slope relationship takes the form shown in Fig. 4. The function F can be approximated, over a limited range, by a power function of z; the exponent is about 1.5 for many natural channels. The corresponding function f depends on the characteristics of the transistor used in the circuit, but in general its approximate power function has an exponent somewhat smaller than 1.5. This is usually of little importance, however, for there is an adjustment available for the slope of the curve and the magnitude of the current at the operating point. Together they can be used to superimpose the curve representing f on that representing F over a reasonable range when the ordinate and abcissa scales have been suitably altered according to the scale factors relating i to Q and e to z. This is shown by the dotted line in Fig. 5.

The Circuit of the Square-Law Resistor.—The basic characteristics of the square-law resistor depend on those of the transistor used. Although all transistors have somewhat similar characteristics, discussion will be centered on those of Sylvania type 2N229, which has proved to be entirely adequate and, in addition, to be the lowest-cost transistor on the market at the user net price of $0.75. This transistor is of the type n.p.n., in which the electrons flow from the emitter towards the collector. A third electrode, the base, receives a small percentage of this current. These currents and the transistor nomenclature are shown in Fig. 6.

When an external voltage, Δe, is applied between the terminals such that B is more positive than A, a current, i_c, will flow from the emitter towards the collector. Its dependence on Δe is similar to the dependence of Q on slope shown in Fig. 4. The current, i_b, shown as being induced by the battery and limited by the resistor r_1, is only about 4% of i_c, but it has a pronounced affect on the latter. Actually, for a given value of Δe, the current i_c can be varied over a range of fifty to one by varying i_b over a similar range, though at a lower magnitude. It should be noted that in this arrangement, the current through resistor r_1 merely circulates and does not flow in any external circuit.

If a resistor r_2 is connected between the base and ground as shown in Fig. 6, an additional current will flow out from the base towards ground if the voltage e at the emitter is negative. As before, the current i_c will be controlled by the base current which now depends on e and r_2 as well as on the battery and r_1. The condenser is practically a short circuit at the frequencies used. Now it can be seen that by adjusting r_1 and r_2 and holding Δe fixed, the current i_c can be made to depend relatively more or less on changes in e or on the fixed current supplied by the battery. Practically, this means that both the slope and the current values can be set at the operating point shown in Fig. 5. In any of these adjustments, it is almost imperative to be able to examine the characteristic curves in their entirety. This can be done with a transistor characteristic curve tracer.

APPENDIX II.—THE EFFECT OF LUMPING CHANNEL PROPERTIES

The important effects of lumping channel properties are shown by taking only the linear properties into account. Then the equivalent electrical circuit can be used together with well-known relationships from electrical engineering.

An artificial, or lumped, transmission line consists of a series of inductance-capacitance elements in the configuration shown in Fig. 7 (resistance is not often large in those instances of interest to electrical engineers, but it is included here because of its increased importance in analog circuits). A progressive wave with a frequency of ω radians per second is modified in traversing the line according to the following expression, wherein e_o is the waveform (as a function of time) at $X = 0$

$$e = e_o \, \epsilon^{-\theta x} \qquad (5)$$

If the transfer constant θ is a pure imaginary of the form jn, n is the phase shift in radians per unit length of line and there is no dissipation. In terms of the inductance and capacitance per unit length of line, n has the value $\omega\sqrt{LC}$. Where resistance is important, it is introduced through the dissipation factor,

$$d_s = \frac{R}{\omega L} \qquad (6)$$

For a single element, occupying a length ΔX, the transfer constant is:

$$\Delta\theta = \tanh^{-1} j\, n_1 \sqrt{\frac{1 - j\, d_s}{1 - n^2 \left(1 - j\, d_s\right)}} \qquad (7)$$

Here the values of L, C, and R are those of this element and $n_1 = \omega\sqrt{L_1 C_1}$ and so on. If the number of elements per unit length is increased without limit and at the same time the values of L, C, and R per element are reduced in proportion, n approaches zero while the transfer constant per unit length becomes

$$\theta = j\, n \sqrt{1 - j\, d_s} \qquad (8)$$

This is the correct expression for a distributed-constant transmission line, the analog of a hydraulic channel.

Returning to Eq. 7, if n_1 is small, but not zero, an approximation can be made by expanding the inverse hyperbolic tangent in an infinite series and then expanding each binomial with the binomial theorum. If terms on the order of n_1^5 are neglected, the result is

$$\Delta\theta = j\, n_1 \left(1 - j\, d_s\right)^{1/2} \left[1 + \frac{n_1^2}{6} \left(1 - j\, d_s\right)\right] \qquad (9)$$

Comparing Eq. 8 with Eq. 9, it is seen that the effect of a finite n_1 is given by the second term in the bracket. Here $\Delta\theta$ is the transfer constant per element and $n_1 = \omega\sqrt{L_1 C_1}$ the value for this element. If there are k elements in a "wavelength" such that $k\Delta\theta = jkn_1 = 2\pi$, it can be seen that to a first approximation (neglecting d_s) n_1 is $2\pi/k$. Thus if k = 8, such that there are 8 sections per wavelength, the error introduced (for zero dissipation) is

$$\frac{n_1^2}{6} = \frac{1}{6}\left(\frac{2\pi}{8}\right)^2 = 0.103 \ldots\ldots\ldots\ldots\ldots (10)$$

If the line is constructed with 8 elements per wavelength, the plus 10.3% error can be compensated for by reducing the values of L, C, and R by the same amount. However, if the signal contains harmonics, they will be affected differently because of the frequency dependence of n, and the wave will be distorted due to the lumping process. Using Eq. 10 for the highest harmonic of interest, k can be selected to limit the error to a tolerable value.

DISCUSSION

G. B. FENWICK,[5] F. ASCE.—It is hoped that Einstein and Harder will, in the future, extend the scope of their very interesting paper to include at least the major results obtained from the electric analog model of the Delta Region, as well as the engineering significance of these results.

The paper explains the use of the analog model in predicting tidal elevations, as well as flow discharges and average velocities at any desired points throughout the maze of Delta channels, both for existing conditions and with the channel system modified by various possible combinations of channel barriers, cutoffs, and enlargements. The further statement in the paper that it is hoped to derive from the predicted flow velocities (cross-sectional averages) the extents of expected salinity intrusions in the Delta channel complex creates some question as to possible methods of accomplishing such derivations.

In many tidal channels the presence of varying degrees of salinity produces density effects that cause the surface and subsurface currents in a given cross-section to be out of phase, and opposite in direction during a considerable portion of each tidal cycle. In such a case, it appears that the average velocity indicated by an electric analog model would be difficult to interpret in terms of total salinity intrusion into the upstream channel complex, particularly when flows in the different interconnected channels involved are also out of phase with each other.

On the other hand, could it be that density effects in the upper portions of the Delta channel are so small that the average velocities obtained from the analog model may, in this particular case, provide a sufficient basis for determining expected salinity intrusion? It would be interesting to obtain comparisons of the results of the electric analog model with those of the hydraulic model of the San Francisco Bay area for the reaches where these two models overlap.

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H. A. EINSTEIN,[6] F. ASCE, and JAMES A. HARDER,[7] M. ASCE.—The writers agree with G. B. Fenwick that it would have been very desirable to have included some of the very interesting results that were obtained from the analog model of the Delta Region in the paper and to have indicated their engineering signigicance. Under an agreement with the California Department of Water Resources, the University of California agreed not to publish these results without their approval. The results of the study will be used in an evaluation of several possible developments within the Delta Region, and while these results will bear on the feasibility of some of the plans, they will not necessarily be decisive.

Members of the ASCE will appreciate that there are many interested groups of citizens, who, supported by the partial results of a feasibility study, would argue powerfully for a particular course of action (in their own economic interest) before all the factors necessary to the decision can be made available to the public.

It is true that within the channels leading to the Delta Region there is sufficiently intense mixing so that the vertical salinity gradients are small. Thus, it might be hoped that the extent of salinity intrusion could be predicted from average tidal velocities. That such predictions are not yet reliable shows that we understand the nature of the mixing process imperfectly, or do not have the computing techniques sufficiently well developed. However, even if quantitative predictions of the change of salinity distributions cannot be made today with sufficient reliability, it is possible to made qualitative predictions. It might be argued that were the upstream area subject to tidal fluctuations reduced, by cutting off some channels for example, the tidal flow in the channels and basins leading to this area would be likewise reduced, with a corresponding reduction in mixing and in salinity intrusion. If a hydraulic model study or an analog study showed that due to changes in the resonance patterns or due to other factors the mean range of the tide had been increased in the upstream area, increased mixing and increased salinity intrusions may be expected. If two such opposing effects leave the tidal flows in the changed channel system the same as they are now, one may draw the conclusion that the salinity conditions would remain the same too. This latter was the actual result for one of the proposed projects.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3278

LONGITUDINAL DISTRIBUTION OF WHEEL LOADS ON A RUNWAY

By R. W. Smith,[1] and R. Horonjeff,[2] F. ASCE

With Discussion by Mr. R. G. Ahlvin

SYNOPSIS

This paper examines existing runways in the light of the jet age, and analyzes the economy factors in runway construction.

INTRODUCTION

Objective.—The objective of this paper was to investigate the manner in which aircraft loads applied to a runway vary in a longitudinal direction. The paper presents a rational basis for determining these loads and their effect on pavement thickness. The maximum applied loads occur at the take-off end of a runway where the aircraft is either stationary, or moving slowly, with little or none of its weight being supported by the wing. As the aircraft accelerates and moves down the runway for take-off, an increasing portion of its gross weight is transferred from the landing gear to its wings. Likewise the weight of an aircraft that is landing is normally less than its weight on take-off. Thus, a portion of the runway does not have to carry the full weight of the aircraft but only a fraction of it.

It is intended to set forth in this report a general method of establishing the loads on the various portions of the runway imposed by take-off and landing

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aircraft. The total load transmitted via the landing gear of an aircraft to the runway pavement at any given instant and at any given location during take-off or landing operations depends on a number of variables. Some of these variables can be computed with some certainty whereas others can only be estimated by judgment and with the background of operational experience as a guide.

Take-Off and Landing Weights.—Obviously, the take-off and landing weights of aircraft will be the basic factors of concern in any pavement design analysis. In developing the method in this paper the maximum take-off gross weight and the maximum structural landing weight, both determined by the aircraft manufacturer, are assumed throughout. Although this assumption does not affect the development of the method, it should be borne in mind that for a specific airplane at a specific airport the maximum take-off weight should not be assumed. Rather, the take-off and landing weights should be estimated as closely as possible, taking into account all factors that affect weight.

The take-off weight for a specific type of aircraft is dependent on (1) the amount of revenue producing load commonly termed payload, (2) the length of the trip, (3) enroute wind and temperature conditions, (4) speed and altitude at which aircraft is to be operated, and (5) amount of reserve fuel carried.

Landing weight equals take-off weight minus the weight of fuel used on the trip. However, on a long trip, the landing weight is so much less than the take-off weight that, for the purposes of pavement design, it would never be the limiting factor. The landing weight that will be considered, as mentioned previously, is the maximum landing weight that is dictated by the structural limitations of the aircraft. The assumption of this maximum weight actually provides a sort of "built-in" safety factor since landing at, or near, this weight would occur under only two conditions: one occurs when a heavily loaded aircraft makes an intermediate stop on a long range flight so that it lands with a very heavy fuel load. This case will account for most high weight landings and will occur at intermediate stations and not at terminal points. The other case of high weight landing would be an emergency when the aircraft is forced to land soon after take-off as a result of some malfunction. In this case fuel jettisoning might be used to reduce the weight rapidly to avoid landing at over the manufacturer's specified maximum landing weight. Landings in this emergency condition are possible but highly infrequent.

Factors Affecting Distribution of Weight Along Runways.—Once the weight of the aircraft is determined there are other factors that will affect the pattern of application of this weight to the runway.

Engine performance.—The rate at which the aircraft accelerates for take-off is dependent, for a given weight, on the engine output or performance. It is an accepted fact of aircraft operation that this performance is not constant, even for a constant temperature, however, under normal circumstances it will not vary by more than 2% or 3%. All values used in this report have been based on operational performance calculated by the aircraft manufacturers at the engine manufacturers' power plant ratings. In applying the analysis to a specific airport it would be necessary to have the aircraft operational data corrected to the proper altitude and air temperature.

Aircraft configuration.—The most important factor of aircraft configuration affecting the distribution of weight along the runway as the airplane accelerates is the airfoil. The lift of the wing at any given speed will be directly proportional to the lift coefficient, a dimensionless factor dependent on the shape of the airfoil. The equation for lift[3] is

$$L = C_1 S q \qquad (1)$$

in which L is the lift in pounds, C_1 denotes the lift coefficient (dimensionless), and q is the dynamic pressure, pounds per square foot, and equals $\frac{\rho v^2}{2}$. Appendix I demonstrates how this relationship is used to determine, for a specific aircraft, that portion of total weight being carried by the wings.

Pilot technique.—Pilot technique during landing determines the point of touchdown and, to some extent, the length of landing rollout. Both of these factors may also be influenced by the location of exit taxiways and the intended destination on the ramp or other part of the airport.

Airport Factors.—For a specific airport location those variables affecting aircraft performance are as follows: the altitude of the airport, the surface wind velocity and direction, temperature, barometric pressure, and runway gradient. For the purpose of this paper the following assumptions are made:

1. Airport altitude of sea level.
2. No wind velocity.
3. Temperature of 59°F.
4. Barometric pressure of 29.92 in. of mercury.
5. Effective runway gradient equals zero.

DEVELOPMENT OF METHOD

Types of Aircraft Considered.—Using the basic information presented in the preceding section it is possible to develop a general method for investigating the manner in which aircraft loads applied to a runway vary in a longitudinal direction. It should be kept in mind that the method can be applied to any aircraft and any airport location. The aircraft considered as examples in this report were chosen to cover a wide spectrum of weight and payload capabilities. All are in use by the major airlines of the United States. They are (a) Convair 340; a twin-engine, propeller driven, (b) DC-6B; a four-engine propeller driven, (c) DC-7; a four-engine, propeller driven, (d) DC-8; a four-engine, jet transport (domestic version), (e) 707-320; a four-engine, jet transport (intercontinental version), and (f) Lockheed Electra; a four-engine, propjet.

The performance characteristics of these aircraft are listed in Table 1.

Load Versus Take-Off Distance.—Fig. 1 has been drawn using the assumptions outlined previously. It shows, for each aircraft, the steadily decreasing load carried by the main gear of the airplane as it accelerates during take-off. In order to arrive at this curve of load vs take-off distance it is necessary to have operational performance at the desired take-off weight for each aircraft. Appendix I outlines in detail for one of the aircraft the steps required to translate this performance data into a pattern of runway loading.

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[3] "Principles of Aerodynamics," by James H. Dwinell, McGraw-Hill Book Co., Inc., New York, 1949.

This pattern was assumed to start at the end of the runway because, in most instances, aircraft make use of the full available length of runway for take-off.

TABLE 1.—CHARACTERISTICS OF AIRCRAFT USED IN ANALYSIS[a]

Characteristic (1)	CV-340 (2)	DC-6B (3)	DC-7 (4)	DC-8 (5)	707-320 (6)	Electra (7)
Maximum take-off weight (lb)	47,000	107,000	122,000	265,000	295,000	116,000
Maximum landing weight (lb)	46,500	85,000	97,000	189,000	195,000	95,650
Maximum take-off gear load	18,000	40,000	46,500	118,000	135,000	52,000
Maximum landing gear load	17,750	32,000	42,000	84,000	89,000	43,000
Lift of speed (knots)	106	116	125	155	158	124
Lift of distance (Feet)	2,500	3,200	3,300	6,600	7,800	3,200
Tire pressure (psi)[a]	69	106	123	150	148	132
Type main gear	Dual	Dual	Dual	Dual Tandem	Dual Tandem	Dual

[a] Approximate values given in table

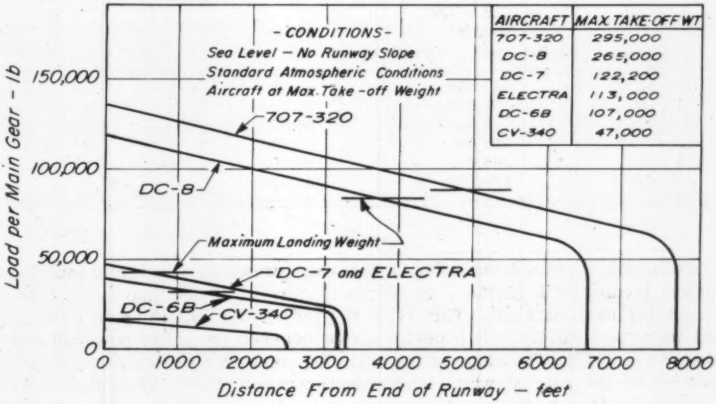

FIG. 1.—MAIN GEAR LOADING DURING TAKE-OFF RUN

The horizontal lines intersecting each load vs take-off distance line indicate the main gear load at the maximum landing weight. It is true that the loads applied to the main gear during landing form an envelope similar to the take-off load vs distance envelope. Contrary to the take-off case, which has a fixed point of departure however, the landing load vs distance envelope cannot be located along the runway with any reliability. A study by the Civil Aeronautics Adminintration has shown that points of touchdown are scattered over at least the first 2,000 ft of runway with no definable pattern other than that the majority are in the first 1,000 ft.[4] Also, as pointed out previously, pilot technique, taxiway location, and eventual airport destination may influence both point of touchdown and roll out distance. For these reasons it may be assumed that practically the entire length of the runway may be exposed to the landing weight. The maximum landing weight may, therefore, be the heaviest load expected on those sections of runway that lie beyond the intersection of the load vs take-off distance curve and the line of maximum landing weight.

Ratio of Maximum Landing Weight to Maximum Take-Off Weight.—Table 2 compares the maximum take-off weight and the maximum landing weight for each aircraft.

It is significant that, in general, the percent difference between these two values increases as the maximum take-off weight increases. This is due to the fact that a greater portion of weight is given to fuel load in the heavier, longer range aircraft.

TABLE 2.—MAXIMUM LANDING WEIGHT AS A PERCENTAGE OF MAXIMUM TAKE-OFF WEIGHT

Aircraft (1)	Maximum take-off weight, pounds (2)	Maximum landing weight, pounds (3)	Percentage of Maximum take-off weight (4)
CV-340	47,000	46,500	99
DC-6B	107,000	85,000	79
Electra	116,000	95,650	82
DC-7	122,000	97,000	79
DC-8	265,000	189,000	71
707-320	295,000	195,000	66

Reduction in Gear Load.—Fig. 2 has been drawn to show the reduction in runway loading with distance in terms of percent of the maximum take-off weight for each aircraft. This type of presentation points up an interesting and important difference in performance between propeller powered and jet powered aircraft. The slope of the unloading curve for each aircraft is an indication of the rate at which load reduction takes place. As is shown in Appendix I, the load reduction at any given point depends on the speed of the aircraft at that point (as accounted for by the dynamic pressure factor in the given equation). The slope of the unloading curve for each airplane is therefore an indication of the acceleration of that airplane. It is obvious from Fig. 2 that the two jet powered aircraft have very similar acceleration characteristics, and that these are very much slower than those of any of the propeller powered aircraft. This fact is well known and of great importance to the aircraft engineer and, as this analysis will show, may also be of importance to the airport engineer.

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[4] Airport Engineering Bulletin No. 1., Civ. Aeronautics Admin., Office of Airports, Washington, D. C.

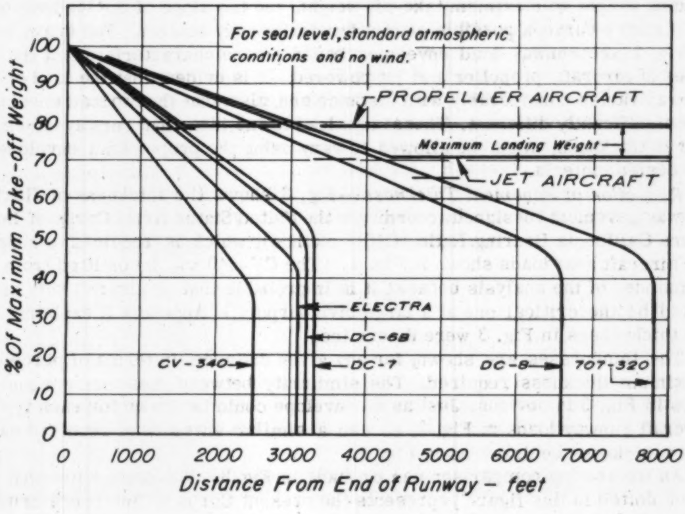

FIG. 2.—RUNWAY LOAD ENVELOPES FOR PROPELLER AND JET-POWERED AIRCRAFT

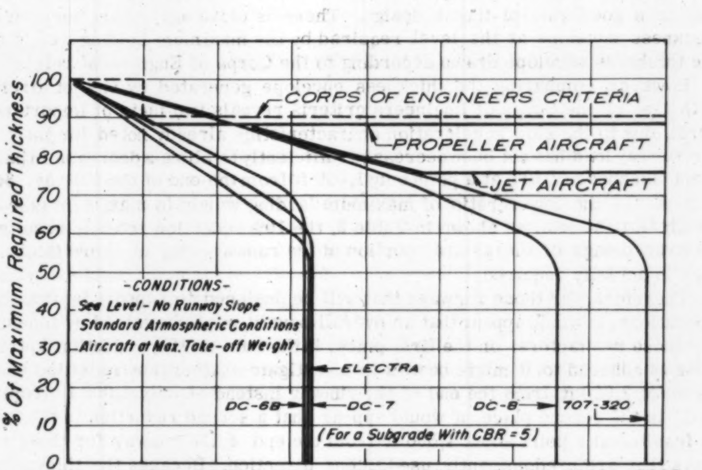

FIG. 3.—RUNWAY THICKNESS ENVELOPES FOR PROPELLER AND JET-POWERED AIRCRAFT

Analyzing the effect of the two factors just noted, the ratio of maximum landing weight to maximum take-off weight, and the slope of the load vs take-off distance curve, a possible pattern for design may be seen. The heavy lines in Fig. 2 are runway load envelopes based on the characteristics of the two types of aircraft, propeller and jet powered. It is evident that the load on the runway varies considerably with distance and, also, that the two load envelopes are significantly different. These facts lead to the idea that runway strength, that is thickness, might be allowed to vary using the proper load envelope as the design criteria.

Reduction in Pavement Thickness.—Fig. 3 shows the thickness of flexible runway pavement (designed according to the United States Army Corps of Engineers California Bearing Ratio (CBR) method), which is required to support the aircraft gear loads shown in Fig. 1. (The CV-340 will be omitted from the remainder of the analysis because it is improbable that an aircraft this small would be the critical one at a large civil airport.) Appendix II explains how the thicknesses in Fig. 3 were determined.

The thicknesses are shown, for any given distance, in terms of percent of maximum thickness required. The similarity between these curves and the ones in Fig. 3 is obvious. Just as an envelope could be drawn for each type of aircraft runway loads in Fig. 2, so can a similar envelope of required pavement thickness be drawn as in Fig. 3.

An interesting comparison can be made in Fig. 3. The dashed line that has been plotted in this figure represents the present Corps of Engineers criteria for thickness. That is, the first thousand feet of runway is designed for maximum static aircraft weight, and the thickness of the central portion of the runway is then reduced by 10%. Fig. 3 shows that for propeller driven aircraft this is a good rule-of-thumb design. There is close agreement between the thickness envelope at the level required by the maximum landing weight and the thickness envelope drawn according to the Corps of Engineers' rule.

However, comparing the thickness envelope generated by the jet aircraft with that of the Corps of Engineers criteria reveals two facts of importance. First, due to the slow acceleration characteristics already noted for the jets, the runway load has not been decreased sufficiently to allow a decrease in pavement thickness of 10% at a point only 1,000 ft from the end of the runway. Second, due to the lower ratio of maximum landing weight to maximum take-off weight for the jets, as shown in Table 2, the 10% reduction actually results in an over-design of the central portion of the runway—that is, more thickness than is actually required.

Therefore, for those runways that will be designed to support jet transport operations, it would appear that an overall benefit might be gained by considering these two factors. In the first place, if the rule of 10% reduction in thickness is adhered to, it might be well to investigate whether this transition should be made 2,000 ft from the end of the runway instead of only 1,000 ft from the end. In the second place, it would appear that a second reduction in thickness is feasible at a point about 4,500 ft from the end of the runway for those runways that are predominantly used in one direction. Because the thickness of pavement required would be based on the heavier jet aircraft, this second reduction would not result in an under-design for the lighter propeller driven aircraft using the same runway. These observations are made on the assumption that instantaneous static loads govern the thickness of the runway. As an aircraft accelerates down a runway, the load on the pavement decreases as the speed increases. The load is therefore not a static one and, as the speed increases, the "effective load" on the pavement, insofar as stresses are concerned, is probably less than the load computed by the method presented in this paper. This point may be worthy of investigation by field tests.

Whether the use of such a design method would result in any economies would depend on the location of the specific airport, the total length of runway required, availability of materials, and so forth. However, it is believed that even if no economies resulted, the placement of material according to this method would result in a more rational and balanced design according to the runway requirements and would therefore be worthwhile.

CONCLUSIONS

As a result of the foregoing analysis, the following conclusions are felt to be valid:

1. The variation of aircraft loads in a longitudinal direction on a runway may be analyzed with sufficient accuracy to provide data for required runway thickness.
2. The rate of decrease in wheel loading of jet-powered aircraft is significantly slower than that for propeller-driven aircraft. This fact should be considered if criteria for variable runway thickness in a longitudinal direction are to be considered.

APPENDIX I.—CALCULATION OF GEAR LOAD VERSUS DISTANCE

Fig. 4 is an example of the airplane performance information for the DC-7 supplied by the manufacturer.[5] This straight-line graph, developed from actual aircraft tests and instrumentation, furnishes the basic data required to determine the load on the main landing gear at any point along the runway for a given aircraft take-off weight. Table 3 shows the preliminary step of determining the distance from the start of take-off. It is possible to enter Fig. 4 with either assumed values of $\frac{V}{2} \times 10^{-3}$ or $\frac{1}{a}$ and to read the other value directly from the graph. Because most aircraft operational data is given in terms of knots, the first step in this example was to choose points every 10 knots, and compute the corresponding aircraft weights. The acceleration distance is computed from

$$S = \frac{1}{a} \frac{V^2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

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[5] "DC-7 Performance Report," Report No. Dev. 1475, Douglas Aircraft Co., Inc., p. 85, November 15, 1953.

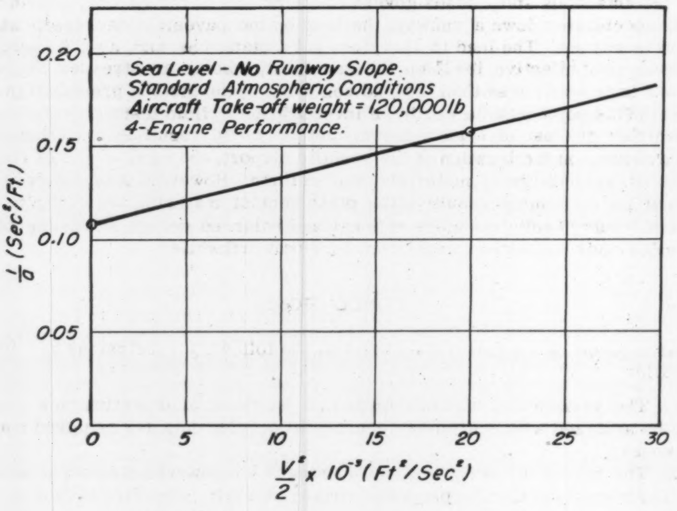

FIG. 4.—DC-7 TAKE-OFF PERFORMANCE

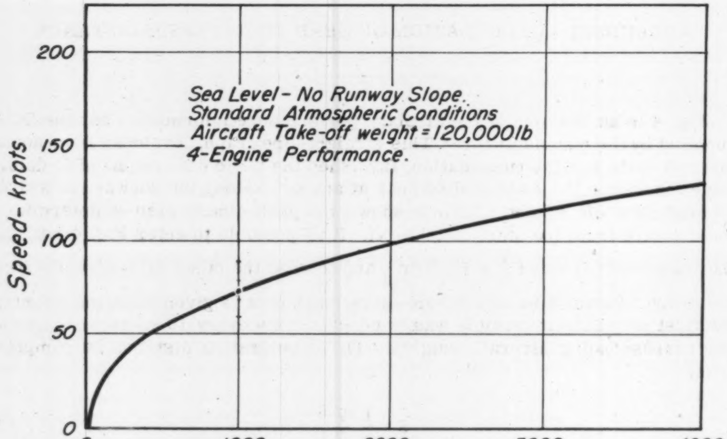

FIG. 5.—DC-7 TAKE-OFF PERFORMANCE

in which S is the distance, in feet, a is the acceleration, in feet per second squared, and V the speed, in feet per second. Speed vs distance is plotted in Fig. 6.

TABLE 3.—DETERMINATION OF DISTANCE FROM START OF TAKE-OFF

Knots (1)	Feet per Second (2)	$\frac{V^2}{2} \times 10^{-3}$ (3)	$\frac{1}{a}$ (4)	Distance, S (5)
0	---	---	---	---
10	16.9	.14	.109	15
20	33.8	.57	.110	65
30	50.7	1.29	.112	145
40	67.6	2.29	.115	265
50	84.5	3.57	.117	420
60	101.4	5.15	.121	625
70	118.3	7.01	.126	885
80	135.2	9.15	.131	1200
90	152.1	11.55	.137	1585
100	169.0	14.30	.144	2060
110	185.9	17.25	.151	2600
120	202.8	20.60	.159	3250
130	219.7	24.10	.168	4050

TABLE 4.—DETERMINATION OF LOAD PER MAIN GEAR

Knots (1)	Lift, pounds (2)	Weight-Lift, pounds (3)	Load per Main Gear, pounds (4)
0	0	120,000	45,500
10	420	119,780	45,300
20	1,690	118,310	44,900
30	3,800	116,200	44,200
40	6,750	113,250	43,100
50	10,550	109,450	41,600
60	15,200	104,800	39,800
70	20,700	99,300	37,700
80	27,000	93,000	35,300
90	34,100	85,900	32,600
100	42,200	77,800	29,600
110	51,100	68,900	26,200
120	60,700	59,300	22,500
130	71,200	48,800	18,500

Knowing the lift coefficient, C_1, for the airfoil, the lift developed by the wing is computed using the formula:

$$L = C_1 \, S \, q \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

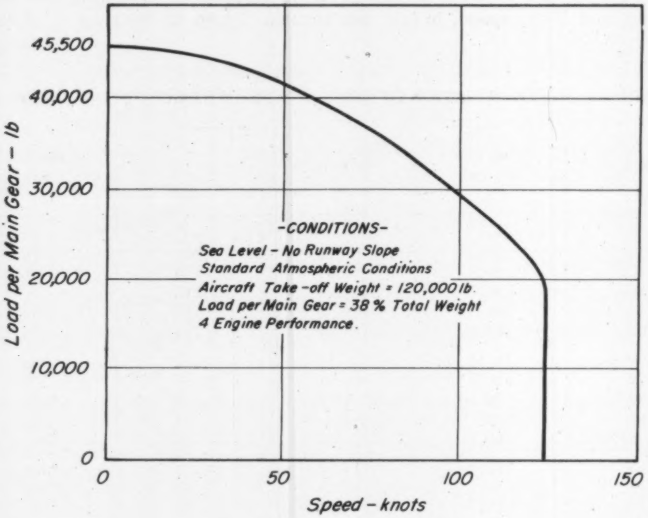

FIG. 6.—DC-7 TAKE-OFF GEAR LOAD vs SPEED

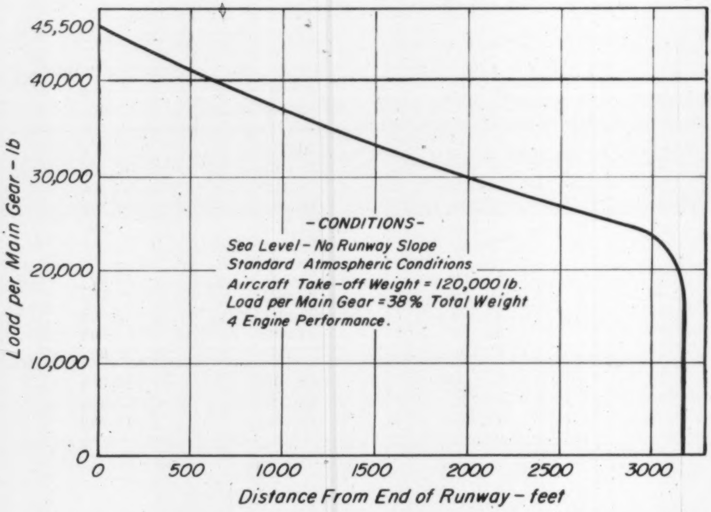

FIG. 7.—DC-7 TAKE-OFF GEAR LOAD vs DISTANCE

in which L is the lift in pounds, C_1 refers to the lift coefficient (dimensionless), S denotes the wing area, in square feet, q is the dynamic pressure $q = 1/2\ p\ V^2$, in pounds per square feet, and p denotes density (0.002378 lb sec^2/ft^4 at sea level).

For the DC-7, C, for take-off configuration is 0.85 and the wing area is 1,462 sq ft. The coefficient of lift C_1, used in the computations, corresponds to the condition of high speed taxiing with all wheels on the pavement. After the nose gear is rotated, the angle of attack increases materially and the coefficient is much higher (e.g. DC-7 $C_1 = 1.52$). Twenty-four percent of the total aircraft weight is carried by the nose gear and 38% of the total carried by each main gear. By subtracting the lift at any speed from the total take-off weight and multiplying this difference by 0.38, the load per main gear is found.

Load per Main Gear vs Speed is plotted in Fig. 7. By cross-plotting Figs. 5 and 6, or by using Table 4, the desired relationship, load per main gear vs distance, is obtained as shown in Fig. 7.

APPENDIX II.—RUNWAY THICKNESS BY THE CORPS OF ENGINEERS METHOD

The Corps of Engineers uses the CBR method in the design of flexible pavements. This method uses an index (CBR) of strength obtained in a penetration-type shear test. This index has been related to a family of curves (CBR design curves) derived from service behavior correlations.[6] Special methods of dealing with the multiple-wheel landing gear configurations of heavy aircraft of the type considered in this report, have also been devised by the Corps of Engineers.[6]

Boeing Airplane Company has published a technical report in which they present a design chart derived from those developed by the Corps of Engineers, that can be used for any combinations of landing gear configuration, wheel loads, subgrade CBR, and tire pressure.[7] Because of the convenience of this presentation, this chart was used in computing thicknesses (at points in multiples of 1,000 ft from the end of the runway). The procedure was as follows: first, the load per main gear was read directly from Fig. 1. Then the ESWL (equivalent single wheel load) and the thicknesses were obtained from the CBR chart shown in the Boeing report.[7] The thickness of flexible pavement was computed for a subgrade CBR of 5. Column 5 in Table 5 shows the relationship of thickness at a given point to the maximum thickness required for a specific aircraft. This information was then plotted in Fig. 3.

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[6] "Development of CBR Flexible Pavement Design Method for Airfields," *Transactions*, ASCE, Vol. 115, 1950.

[7] "Airport Strength Requirements Boeing Model 707 Jet Transport," Document No. D-17322, Boeing Airplane Co., September 30, 1955.

TABLE 5.—DESIGN TABLE

	Distance[a] (1)	Load per Main Gear, pounds (2)	ESWL[b] (3)	Thickness, inches (4)	Percentage of Maximum Thickness (5)
Propeller Driven Aircraft					
DC-6B	0	40,000	26,400	24	100
	1000	34,000	21,500	21	89
	1300[c]	32,000	20,000	21	
	2000	27,500	16,800	18	75
	3000	21,200	12,300	16	68
DC-7	0	46,500	46,000	26	100
	750[c]	42,000	28,200	24	92
	1000	39,000	25,500	23	89
	2000	32,000	20,000	21	81
	3000	25,500	15,500	18	69
Electra	0	51,000	42,000	30	100
	1000	40,500			
	1500	37,000	26,500	24	80
	2000	32,500			
	3000	28,500	16,000	18	60
Jet Powered Aircraft					
DC-8	0	118,000	65,000	38	100
	1000	109,000	58,000	36	95
	2000	99,000	52,000	34	90
	3000	89,000	45,000	32	84
	3700[c]	84,000	41,000	30	79
	4000	80,000	38,500	29	76
	5000	72,000	31,000	26	69
	6000	64,000	24,000	23	61
707-320	0	135,000	70,000	40	100
	1000	125,000	62,500	38	95
	2000	116,000	57,000	36	90
	3000	107,000	50,500	34	85
	4000	98,000	45,000	32	80
	4800[c]	89,000	39,500	29	73
	5000	88,000	39,000	29	73
	6000	78,000	34,000	27	68
	7000	69,000	28,500	25	63

[a] Distance in feet from start of roll.
[b] ESWL: Equivalent single wheel load—computed by Corps of Engineers procedure.
[c] Point at which load per main gear is same as at maximum landing weight.

DISCUSSION

R. G. AHLVIN.[8]—The authors are to be congratulated for their treatment of a topic which is significant in airfield pavement design and on which a report has not previously been published. The paper is very interesting and worthy of study by those concerned with airfield pavements.

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[8] U. S. Army Corps of Engrs., Waterways Experiment Sta., Vicksburg, Miss.

Since the United States Corps of Engineers', Dept of the Army, design requirements are used for some of the authors' comparisons, a few comments on the development of those requirements may be in order. The 10% reduction in thickness was adopted by the Corps of Engineers for portions of runways between the 1,000-ft ends because of failures that had been experienced in the ends of constant-thickness runways.

Following recognition of this failure trend, studies similar to those reported in this paper were made in attempts to determine why the ends of runways should fail earlier than the center sections. It was concluded that wing lift, increasing with increase in velocity, while obviously a contributing factor, did not completely account for the runway-end failure phenomenon.

The authors, in their final paragraphs, recognize a factor other than lift which plays an important part in design thickness requirements—the relation between "static" and "moving" loadings. Tests conducted[9,10] by the Corps of Engineers at Marietta, Georgia, in 1945, and at Stockton, Calif., during 1945-1947 clearly indicated significantly smaller stresses and deflections beneath slowly moving loads than beneath static loads.

The Corps of Engineers' design thickness reductions for interiors of runways are based on the combined effect of lift and moving loadings. At the time the reductions were originally incorporated in the design, a 20% reduction was considered, but only a 10% reduction was adopted. Thus, for propeller-type aircraft, the 10% thickness reductions are somewhat conservative. The reported study, in showing jet aircraft to have a lesser portion of their weight airborne 1,000 ft from the runway end, indicates that the 10% thickness reduction is less conservative for jet aircraft than for propeller aircraft. It does not indicate an unconservative element in current design criteria.

In a number of instances, airfields constructed by the Corps of Engineers have been subjected to significant amounts of traffic by heavy jet aircraft. Condition surveys have in no case indicated pavement distress initiating in the second 1,000 ft of runway on any of these airfields.

The reported study does point the way to possible further reductions in thickness in the central portion of very long runways. This possibility has already been recognized and is not being ignored.

As an example of progress in effecting economies in regard to traffic distribution, there is another traffic factor which can be considered in the construction of large airfields for heavy aircraft. Recent Corps of Engineers' criteria for very heavy load airfields have taken advantage of this additional consideration. It concerns the lateral distribution of traffic. Pavements for occasional use can be significantly thinner than those for regular use, and the lateral distribution of aircraft on a wide runway is such that strips of the runway one-third of its width and along each edge are rarely, if ever, loaded by using aircraft. For heavy aircraft, the reduced thicknesses in these edge strips are still adequate for service and snow removal equipment, and current Corps of Engineers' criteria permit reduced strength designs in the 100-ft-edge strips of 300-ft heavy-duty runways. R. Horonjeff, in joint effort with J. H. Jones, reported a study of this nature.[11]

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[9] "Certain Requirements for Flexible Pavement Designs for B-29 Planes," U. S. Corps of Engrs., Waterways Experiment Sta., Tech. Memorandum, August, 1945.

[10] "Accelerated Traffic Test at Stockton Airfield, Stockton, Calif.," U. S. Corps of Engrs., Sacramento Dist., (Stockton Test No. 2), May, 1948.

[11] "The Effect of Traffic upon Runway Pavement Cross Section," by R. Horonjeff and John Hugh Jones, Proceedings Separate No. 720, ASCE, Vol. 81, June, 1955.